1958 REGIONAL MEETING—DETROIT—OCTOBER 27-29, 1958

JULY 1958
No. 1
VOL. 30

JOURNAL
OF THE
AMERICAN CONCRETE INSTITUTE

Papers and Reports 1-150

55-1 **Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)** ..... ACI COMMITTEE 208 — 1

55-2 **Recommended Practice for Design of Concrete Pavements (ACI 325-58)** ..... ACI COMMITTEE 325 — 17

55-3 **Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)** ..... ACI COMMITTEE 617 — 53

55-4 **Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)** ..... ACI COMMITTEE 711 — 83

55-5 **Origin, Evolution, and Effects of the Air Void System in Concrete. Part 1—Entrained Air in Unhardened Concrete.** RICHARD C. MIELENZ, VLADIMIR E. WOLKODOFF, JAMES E. BACKSTROM and HARRY L. FLACK — 95

55-6 **Load Test on Flat Slab Floor with Embedded Steel Grillage Caps** ..... DONALD D. MEISEL, CYRIL D. JENSEN and WALTER H. WHEELER — 123

55-7 **Heavy Media Processing of Gravels in New Brunswick** ..... I. D. MacKENZIE — 133

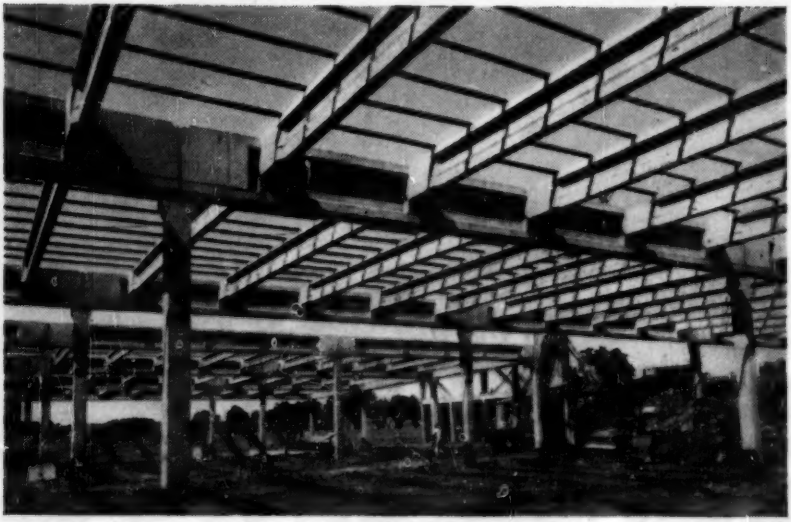
55-8 **Load Test of 120-Ft Precast, Prestressed Bridge Girder** ..... FAZLUR R. KHAN and ANDREW J. BROWN — 139

Current Reviews 151-160

News Letter 1-34

ACI
PROCEEDINGS
VOL. 55

STRESSING
A STRUCTURAL TRIUMPH



● Sanitation and cleanliness, paramount in food processing, are best attained with precast, prestressed concrete, providing wide-open floor space and smooth surfaces that keep that new look, are easy to clean, functionally attractive, corrosion-proof, fire-safe and durable—designed for today's needs, yet built for the years ahead.

Noteworthy is this large modern structure which houses the multiple food processing facilities of Charles Dennery, Inc., in Jefferson Parish, Louisiana. The well-designed building covers 72,000 square feet and contains the plant's varied operations—converting raw materials into packaged food products.

Beams, girders, purlins, and columns were supplied and erected by Belden Concrete Products, Inc., of New Orleans. A quick start and early completion were obtained with 'Incor'*, making possible assembly-line production through fast stripping and re-use of forms—advantages always assured with America's FIRST high early strength portland cement.

*Reg. U. S. Pat. Off.

LONE STAR CEMENT CORPORATION

Offices: ABILENE, TEX. • ALBANY, N. Y. • BETHLEHEM, PA. • BIRMINGHAM • BOSTON • CHICAGO • DALLAS • HOUSTON • INDIANAPOLIS
KANSAS CITY, MO. • LAKE CHARLES, LA. • NEW ORLEANS • NEW YORK • NORFOLK • RICHMOND • SEATTLE • WASHINGTON, D. C.

LONE STAR CEMENT, WITH ITS SUBSIDIARIES, IS ONE OF THE WORLD'S LARGEST CEMENT PRODUCERS. 21 MODERN MILLS. 48,900,000 BARRELS ANNUAL CAPACITY

JULY 1958

No. 1

Vol. 30

PROCEEDINGS VOL. 55

JOURNAL
OF THE
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute under the authority of its Board of Direction

President

DOUGLAS McHENRY

Vice-Presidents

PHIL M. FERGUSON — JOE W. KELLY

Directors

ARSHAM AMIRIKIAN — BRYANT MATHER
WILLIAM H. ARMSTRONG — RAYMOND C. REESE
S. J. CHAMBERLIN — LEWIS H. TUTHILL
CLAYTON L. DAVIS — C. D. WAILES, JR.
BRUCE E. FOSTER — STANTON WALKER
ROBERT C. JOHNSON — CEDRIC WILLSON

Past Presidents

CHARLES S. WHITNEY
FRANK KEREKES — WALTER H. PRICE

Secretary-Treasurer

WILLIAM A. MAPLES

Technical Director
PAUL F. RICE

Managing Editor
ROBERT E. WILDE

Associate Editor
ROBERT G. WIEDYKE

Advertising Manager
CHARLES L. COUSINS

Manager, Publication Sales
ISABELLE DAVIS

Membership Secretary
EVELYN SCHLITZ

Cost per copy $1.50. Extra copies to ACI members $1.00.

Papers, committee reports, and other contributions to the **Proceedings** pages of this Journal, as submitted in furtherance of Institute objectives, are, if published, accepted on the sole responsibility of their authors. Institute authority attaches only to standards adopted as provided in the Bylaws.

Advertising. See statement under "Alphabetical List of Advertisers," final page **News Letter section.**

Membership dues and Journal subscription rates. Members receive the Journal as one of the considerations of dues payment. Membership grades are: Individual (U. S., its possessions, and North and Central America), $15, Individual (all other foreign countries), $12, Corporation, $50, Contributing, $100, Junior, $7.50, Student, $5. Subscription price is $18 per year, payable in advance. The Journal is issued monthly, July to June (the volume year being completed by title page, contents, indexes, and closing discussion in Part 2 of the subsequent September and December issues).

Bound volumes 1 to 53 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1957) are for sale as far as available at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

Address: American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich. **Copyright©, 1958,** American Concrete Institute. Printed in U. S. A. Entered in the Post Office at Detroit, Mich., as mail of the second class under provisions of the Act of Mar. 3, 1879.

the ACI JOURNAL

is edited by the Secretary of the Technical Activities Committee under the direction of the Committee

WALTER J. McCOY
Chairman

DOUGLAS McHENRY
(ex officio)

WILLIAM A. MAPLES
Secretary

G. E. BURNETT

E. A. FINNEY

CLYDE E. KESLER

BRYANT MATHER

MICHAEL V. PREGNOFF

L. H. TUTHILL

WILLIAM A. MAPLES
Editor

ROBERT E. WILDE
Managing Editor

MARY K. HURD
Associate Editor

MARIAN PATTEN
Editorial Assistant

RUTH GABLE
Editorial Secretary

Coming next month in the JOURNAL

Two ACI technical committee reports will be published in the August JOURNAL:

"Curing Concrete" submitted by ACI COMMITTEE 612

"Pressures on Formwork" prepared by ACI COMMITTEE 622

●

Five papers originally presented at the Fatigue Symposium of the ACI 54th annual convention, February, 1958, will also appear:

"Fatigue of Concrete—A Review of Past Research" by GENE M. NORDBY

"Effect of Range of Stress on Fatigue Strength of Plain Concrete Beams" by JOHN W. MURDOCK and CLYDE E. KESLER

"Probability of Fatigue Failure of Plain Concrete" by JOHN T. MCCALL

"Fatigue Behavior of Reinforced Concrete Beams" by TIEN S. CHANG and CLYDE E. KESLER

"Fatigue Properties of Concrete Beams" by THOMAS E. STELSON and JOHN N. CERNICA

●

JAMES E. BACKSTROM, RICHARD W. BURROWS, RICHARD C. MIELENZ, and VLADIMIR WOLKODOFF are authors of "Origin, Evolution, and Effects of the Air Void System in Concrete. Part 2—Influence of Type and Amount of Air-Entraining Agent."

●

"Design of Symmetrical Columns with Eccentricities in One or Two Directions" is the work of FREDERICK P. WIESINGER.

●

G. C. ERNST has submitted results of "Tests on Moment and Shear Redistribution in Two-Span Continuous Reinforced Concrete Beams."

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

Secretary, Technical Activities Committee
AMERICAN CONCRETE INSTITUTE
P. O. BOX 4754, REDFORD STATION, DETROIT 19, MICHIGAN

Title No. 55-1

Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)*

Reported by ACI Committee 208

C. P. SIESS
Chairman

ARTHUR P. CLARK**
PHIL M. FERGUSON
H. J. GILKEY
W. H. JACOBS
R. C. REESE
DAVID WATSTEIN
A. C. WEBER
C. A. WILLSON

SYNOPSIS

This test procedure provides a uniform basis for comparison of bond qualities of different reinforcing bars. The recommended test method uses companion beam type specimens, cast horizontally in inverted and normal positions so that the effects of water gain and settlement of concrete are included in the evaluation. The concrete surrounding the bars is in tension as bars are ordinarily used, which makes the results more acceptable to some users. Minimum criteria for acceptance are not included since the purpose is merely to establish relative bond values for the different bars under consideration.

The test may be used to evaluate the effectiveness of deformed bars having characteristics other than those described in "Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement," ASTM A 305-56T. For construction under the ACI Building Code, an evaluation of such bars is necessary since the code definition for a deformed bar is one which meets the requirements of ASTM A 305. Manufacturers of bars meeting the requirements of ASTM A 305 may use the test procedure for product improvement and to evaluate modifications in the design of deformations.

INTRODUCTION

Previously published identically as information,† this test procedure had a thorough workout through its use in extensive testing programs culminating in the evolution of specifications covering minimum geometric requirements.‡ Acceptance by the ACI Building Code committee§ of the ASTM A 305 definition of a deformed bar produced an immediate drastic change in both structural

*Adopted as a standard of the American Concrete Institute at its 54th annual convention, Feb. 25, 1958, as reported by Committee 208; ratified by letter ballot June 1, 1958. Title No. 55-1 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each to ACI members ($1.00 to nonmembers). **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†ACI JOURNAL, Feb. 1945, *Proc.* V. 41, pp. 273-292; discussion, Nov. Supplement 1945, *Proc.* V. 41, pp. 292-1 to 292-8.

‡ASTM Designation A 305.

§ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-51)," ACI JOURNAL, Apr. 1951, *Proc.* V. 47, pp. 589-652.

**Deceased.

and general practice, resulting in the almost immediate disappearance of old-style nonconforming deformed reinforcement.

Background considerations involving alternative possibilities, controversial aspects, and special credits due to individuals and organizations are covered in the foreword and discussions that accompany the 1945 publication of the test procedure.*

BEAM SPECIMENS

1. There shall be 18 beam specimens, the details of which are shown on the accompanying Drawings 1 and 2.†

2. The nine A, B, and C specimens (three for each length of embedment) shall be cast erect as shown. The nine AA, BB, and CC specimens shall be cast inverted.

CONCRETE

3. The concrete shall have a slump of 5 to 6 in. in the standard slump test, and an ultimate strength of 3500 psi (plus or minus 300 psi) at age of 28 days. Preliminary tests shall be made to determine the proportions of the prescribed materials required to develop this ultimate strength. The cement shall be equal parts by weight of four standard brands of Type I portland cement. The fine aggregate shall be natural sand well graded from 0 to No. 4 sieve. The coarse aggregate shall be gravel, all of which shall pass either a sieve having ¾ in. square openings or a screen with 1 in. diameter openings. All materials shall conform to ASTM standards.

BARS

4. The deformed or plain bars to be tested for bond shall be nominally ⅞ in. in diameter. The weight shall be measured and reported in lb per ft to the nearest 0.01 lb. In the interest of uniformity and simplicity, values for tensile stress, yield point, bond stress, and modulus of elasticity will be calculated on the basis of the cross-sectional area and the perimeter listed in handbooks for a plain round bar of the same nominal diameter. Specifically the cross-sectional area of a ⅞-in. deformed or plain bar (meeting ASTM requirements for weight tolerances) will be taken as 0.60 sq in. and the perimeter as 2.75 in.

5. The pull at yield point and the relation between pull on bar and elongation shall be determined from several typical samples of each design of bar. This relation will serve to indicate the pull on bar at various stages of the test from the strain measurements to be made at the sides of the bar at notches cast in the beam specimens.

6. The bars submitted for test shall be representative of the product of the mill which submits them, and shall not be given any treatment to produce

*ACI JOURNAL, Feb. 1945, *Proc.* V. 41, pp. 273-277, and Nov. Supplement 1945, *Proc.* V. 41, pp. 292-1 to 292-8.

†The 11 drawings, pp. 6-16, which constitute an important part of the test procedure are available in blueprints (22 x 35 in.) *in complete sets only* at $36.00 per set from ACI headquarters.

an abnormal surface. The surface of all bars, however, shall be thoroughly cleaned with naphtha or carbon tetrachloride just before insertion in mold to remove all oil or grease and any other foreign matter. If, in any case, there is a question as to whether the bars submitted are truly representative, new samples shall be selected for the tests by an experienced disinterested inspector.

MOLDS AND ASSEMBLIES

7. The mold shall be of metal, and shall be coated to prevent adhesion to the concrete and sealed to prevent leakage. The mold assembly is shown in Drawing 4 and the exterior parts of mold are detailed in Drawing 5. Two notches 6 in. long and 3 in. high shall be cast in all beam specimens, as shown in Drawings 1, 3, and 4, and the insert for these notches shall be as detailed in Drawing 6.

8. Details of the main reinforcing bar and stirrup assembly ready for installing in the mold are shown in Drawings 10 and 11.

9. The $\frac{7}{8}$-in. bar shall be so placed in the mold that the parting line of its deformation pattern (that is, the parting line of upper and lower halves of bar as rolled) shall come at the sides (see detail, Drawing 4). The 3-in. extensions of the bar beyond the center lines of the reactions shall be covered with a rubber sleeve so as to insure no bond. The stirrups shall be $\frac{1}{2}$-in. round deformed bars, furnished by the maker of the $\frac{7}{8}$-in. bars to be tested, and shall be welded into loops as shown in Drawing 11. All steel reinforcement shall be rigidly held in place in the mold (see Drawing 4).

10. Within the length of each notch, two yokes, similar to those shown in Drawing 3 and detailed in Drawing 7 shall be clamped into carefully drilled contact points on 5-in. centers on each side of the bar. These points shall be so located (by means of the jig detailed in Drawing 10) as to make the two yokes as nearly parallel as is practicable. The pull or the fiber stress in the bar shall be determined from the average of the change in distance between the two yokes.

CASTING, REMOVAL OF FORMS, CURING

11. The three identical specimens representing one length of embedment of a given bar shall be made on different days to average minor differences in the mix and other variables. The concrete shall be deposited in two layers and each layer shall be sliced and rodded to insure complete contact with the steel reinforcement. Immediately after placing, the concrete should be struck off level with the top edge of the mold with a minimum of disturbance. The specimens must not be exposed to any vibrations or other disturbances during the first 24 hr after casting.

12. At least three standard 6 x 12-in. cylinders for compression tests shall be made from the various batches of concrete mixed for each beam, one cylinder from any one batch.

13. After preliminary curing in the mold for 5 days and removal of molds as indicated below, the beam specimens and cylinders shall be stored in a standard fog room at a temperature of 65 to 75 F and tested moist at age of 28 days. While in the molds the beams and cylinders shall be protected against loss of moisture from the concrete.

14. To avoid damaging the beam specimens the sides and ends of the molds and the tapered inserts shall not be removed earlier than 5 days after placing the concrete. Care shall be exercised to avoid damaging the green specimens during removal to moist room. To this end it is recommended that the beams be placed on stiff wood planks or equivalent supports during trucking operations. The pieces forming the notches in the beam are to be withdrawn at the end of the 5-day period in the manner shown in Drawing 6.

TESTING

15. The sides of the specimen shall be whitewashed about 2 hr before test and allowed to dry so that the sequence and development of cracks may be easily marked by pencil lines which will be visible on any photographs taken after test.

16. Loads shall be applied through rollers or rockers. The beams shall rest on rocker or roller supports to avoid restraint to elongation of bottom fibers of beam as load is applied. The apparatus for supporting and applying load to beam specimen is shown in Drawing 3 and detailed in Drawing 9.

17. A reference steel angle clip shall be screwed to the concrete of the beam sides as near as is convenient to the notch and on the reaction side of the load as shown in Drawing 3 and detailed in Drawing 7. Between this angle and the nearer extensometer yoke the slip of bar immediately under the load shall be measured. This slip will be designated the slip at "loaded" end of bar.

18. The slip of bar at end of beam shall be measured in the manner shown in Drawing 3 with the gage detailed in Drawing 7. This slip will be designated as the slip at "free" end of bar.

19. The deflection at center of the beam shall be measured in the manner shown in Drawing 3 with the apparatus detailed in Drawing 8.

20. The rate of load application during the beam tests shall not be greater than 2000 lb per min nor greater than is consistent with securing reliable test data. Readings may be secured for continuous application of load if available manpower permits. With a shortage of observers it may be preferable either to stop the machine for securing readings at each increment of load or not to stop the machine but to read only half the gages as a given load is passed; reading the other half of the gages at the next increment. When readings are staggered in this manner the indications of corresponding pairs of gages on opposite sides of the beam should be noted simultaneously as in the case of gages indicating slip and strain at a given notch.

21. Load shall be applied to failure, and the report shall state whether failure of the beam was due to failure in bond, diagonal tension, failure of bottom bar in tension or of top concrete in compression, or by splitting of concrete as a result of failure in bond. Cracks due to settlement of concrete will normally occur in the top surface of the beam specimens AA, BB, and CC cast inverted. These should be noted and diagrammed for later reference.

22. Readings of slip at both free and loaded ends of the bar, of pull in the bar, and of deflection of the beam at center shall be taken in such number (preferably not less than 15) as to give smooth curves in graphical representations of the test data.

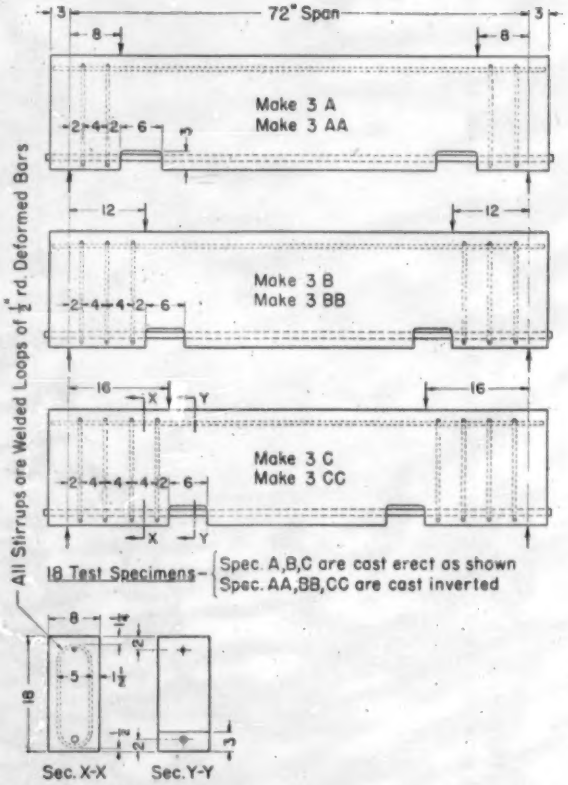
REPORT

23. Curves showing slip of bar shall have the amounts of slip as abscissae, and the beam loads, pulls or fiber stresses in bar, unit bond stresses, and values of u/f_c' as ordinates. The unit bond stress u is the value obtained by dividing the indicated pull on the bar in pounds (from strain measurements) by the area in bond. The area in bond is obtained by multiplying the nominal perimeter of 2.75 in. for the $\frac{7}{8}$-in. diameter bar (see Paragraph 4) by the effective length of embedment (8, 12, or 16 in.). Load-deflection curves shall show the amount of deflection as abscissae and beam loads as ordinates.

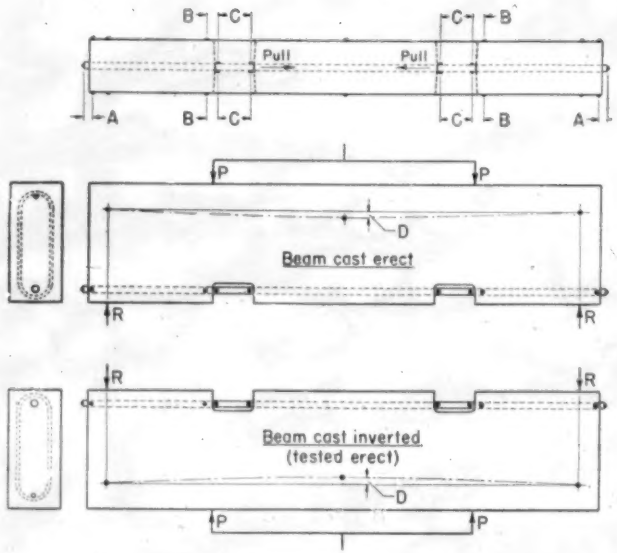
24. The report shall contain a detailed description of the testing setup, and of all devices and measuring instruments employed.

Discussion of this report should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

See Drawings 1-11 on the following pages: pp. 6-16. Blueprints (22 x 35 in.) of the 11 drawings are available in complete sets only at $36.00 per set from ACI headquarters.



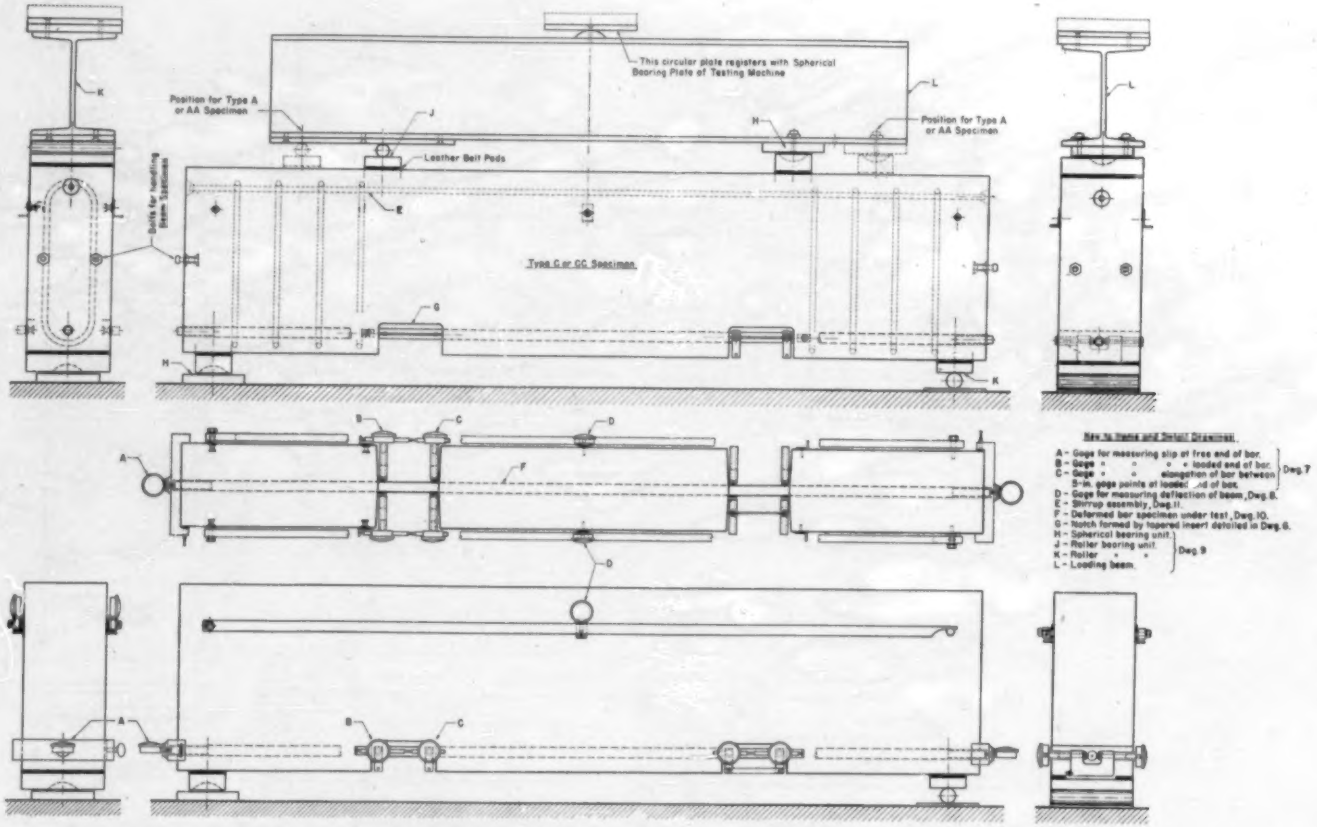
Dwg. 1—Beam specimens for determining the "relative bond value" of reinforcing bars with various types of deformations



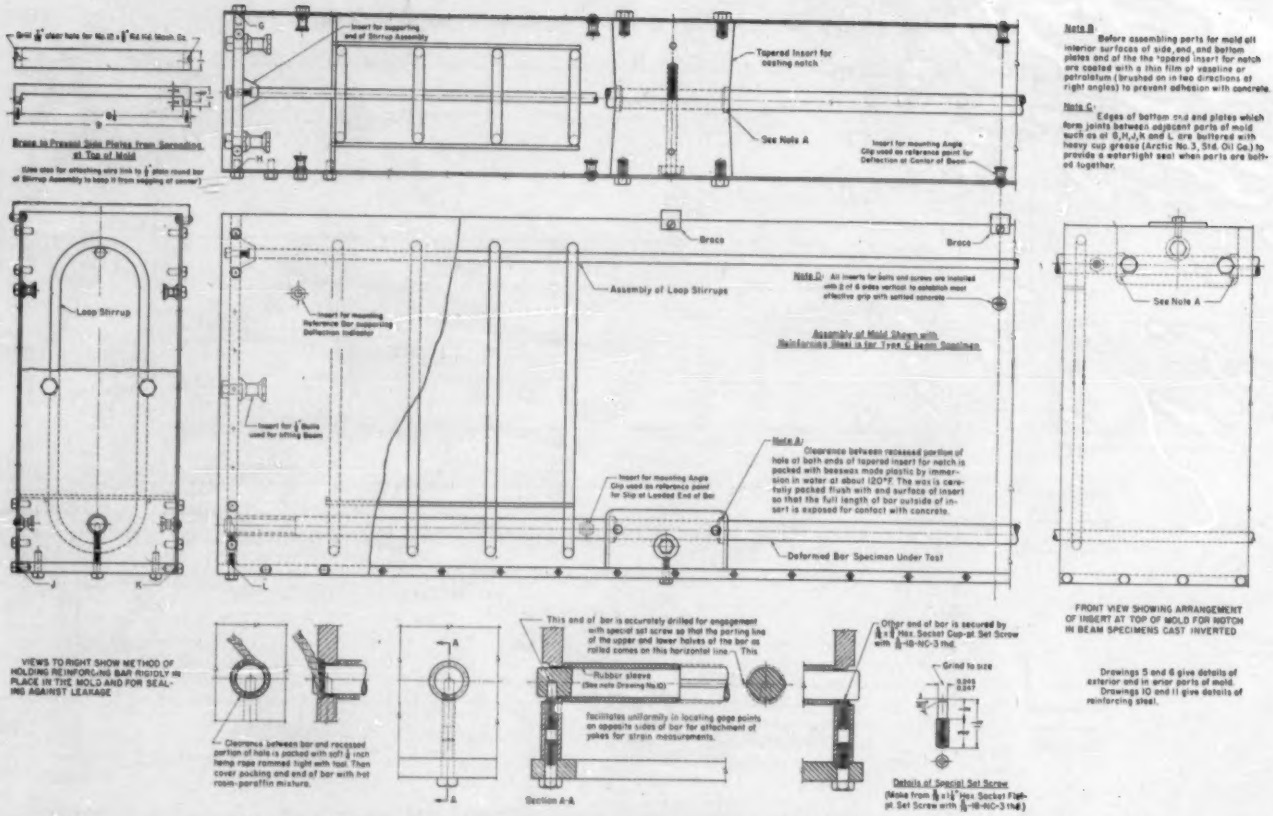
Reference Points

A - Slip at free end of bar
B - Slip at loaded end of bar
C - Elongation of bar between 5" gage points at loaded end
D - Deflection at center of beam near neutral surface
P - Applied load
R - Reaction to applied load

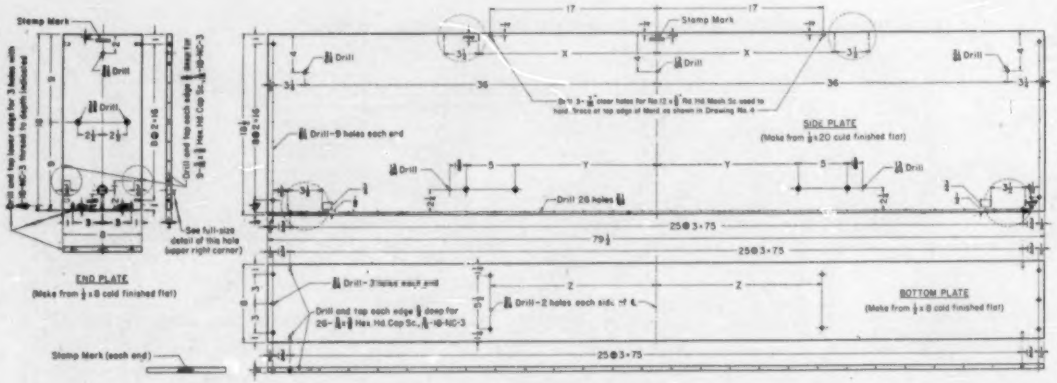
Dwg. 2—Diagrams showing reference points for measurements on beam specimens


This circular plate registers with Spherical Bearing Plate of Testing Machine
Position for Type A or AA Specimen
Position for Type A or AA Specimen
Leather Belt Pads
Bolts for handling Beam Specimen
Type C or CC Specimen
Key to Name and Detail Drawings
A - Gage for measuring slip at free end of bar.
B - Gage " " " " loaded end of bar.
C - Gage " " " elongation of bar between 5-in. gage points at loaded end of bar. } Dwg. 7
D - Gage for measuring deflection of beam, Dwg. 8.
E - Stirrup assembly, Dwg. 11.
F - Deformed bar specimen under test, Dwg. 10.
G - Notch formed by tapered insert detailed in Dwg. 6.
H - Spherical bearing unit.
J - Roller bearing unit. } Dwg. 9
K - Roller " "
L - Loading beam.

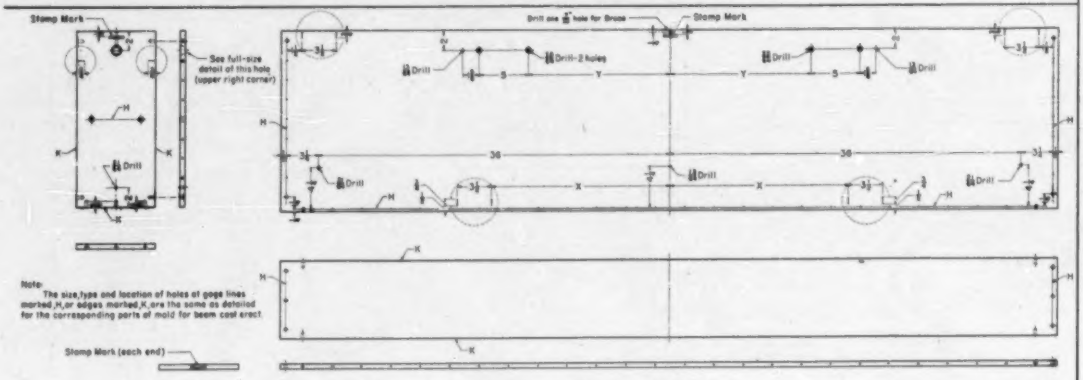
Dwg. 3—General drawing of beam specimen ready for test with all gages in position


Insert for supporting end of Stirrup Assembly
Tapered Insert for casting notch
See Note A
Insert for mounting Angle Clip used as reference point for Deflection at Center of Beam
Brace
Brace
Loop Stirrup
Assembly of Loop Stirrups
Insert for mounting Reference Bar supporting Deflection Indicator
Insert for 1/2 Bolts used for lifting Beam
Assembly of Mold Shown with Reinforcing Steel in for Type C Beam Specimen
Insert for mounting Angle Clip used as reference point for Slip at Loaded End of Bar
Deformed Bar Specimen Under Test
Note D: All inserts for bolts and screws are installed with 2 of 6 sides vertical to establish most effective grip with settled concrete
Note A: Clearance between recessed portion of hole at both ends of tapered insert for notch is packed with beeswax made plastic by immersion in water at about 120°F. The wax is carefully packed flush with end surface of insert so that the full length of bar outside of insert is exposed for contact with concrete.
Braze to Prevent Side Plates from Spreading at Top of Mold
(Use also for attaching wire link to 1/2 plate round bar of Stirrup Assembly to keep it from wagging at corner)
Note B: Before assembling parts for mold all interior surfaces of side, end, and bottom plates and of the tapered insert for notch are coated with a thin film of vaseline or petrolatum (brushed on in two directions at right angles) to prevent adhesion with concrete.
Note C: Edges of bottom end and plates which form joints between adjacent parts of mold such as at B, M, N and L are buttered with heavy cup grease (Arctic No. 3, Std. Oil Co.) to provide a watertight seal when parts are bolted together.
See Note A
FRONT VIEW SHOWING ARRANGEMENT OF INSERT AT TOP OF MOLD FOR NOTCH IN BEAM SPECIMENS CAST INVERTED
VIEWS TO RIGHT SHOW METHOD OF HOLDING REINFORCING BAR RIGIDLY IN PLACE IN THE MOLD AND FOR SEALING AGAINST LEAKAGE
Clearance between bar and recessed portion of hole is packed with soft 1/4 inch hemp rope rammed tight with tool. Then cover packing and end of bar with hot rosin-paraffin mixture.
Section A-A
This end of bar is accurately drilled for engagement with special set screw so that the parting line of the upper and lower halves of the bar as rolled comes on this horizontal line. This
Rubber sleeve (See note Drawing No. 10)
facilitates uniformity in locating gage points on opposite sides of bar for attachment of yokes for strain measurements.
Other end of bar is secured by 1/2 x 1 1/2 Hex. Socket Cup-pt. Set Screw with 1/2-13-NC-3 thd.
Grind to size
0.065
0.067
Details of Special Set Screw
(Made from 1/2 x 1 1/2 Hex. Socket Flat-pt. Set Screw with 1/2-13-NC-3 thd.)
Drawings 5 and 6 give details of exterior end and ends of mold.
Drawings 10 and 11 give details of reinforcing steel.

Dwg. 4—Assembly of mold with reinforcing steel ready to receive concrete


Stamp Mark
END PLATE
(Make from ½ x 8 cold finished flat)
SIDE PLATE
(Make from ½ x 20 cold finished flat)
BOTTOM PLATE
(Make from ½ x 8 cold finished flat)
Drill 28 holes
Drill-9 holes each end
Drill-2 holes each side of ℄
Drill-3 holes each end
See full-size detail of this hole (upper right corner)
Stamp Mark (each end)
25@3=75
79½

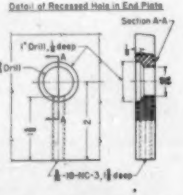
EXTERIOR PARTS OF MOLD FOR BEAMS CAST ERECT (Specimens A,B,C)


Stamp Mark
Drill one ⅝ hole for Brace
See full-size detail of this hole (upper right corner)
Drill-2 holes
Stamp Mark (each end)

Note: The size, type and location of holes at gage lines marked H, or edges marked K, are the same as detailed for the corresponding parts of mold for beam cast erect.

EXTERIOR PARTS OF MOLD FOR BEAMS CAST INVERTED (Specimens AA,BB,CC)

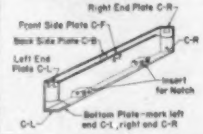
Detail of Recessed Hole in End Plate



Values of Dimensions X, Y, Z for Beam Molds

Specimen	Casting Position	Length of Embedment	X	Y	Z
A	Erect	8	26½	22½	25
AA	Inverted	8	"	"	"
B	Erect	12	22½	18½	21
BB	Inverted	12	"	"	"
C	Erect	16	18½	14½	17
CC	Inverted	16	"	"	"

Marking System for Identifying Parts of Mold for a Given Beam Specimen

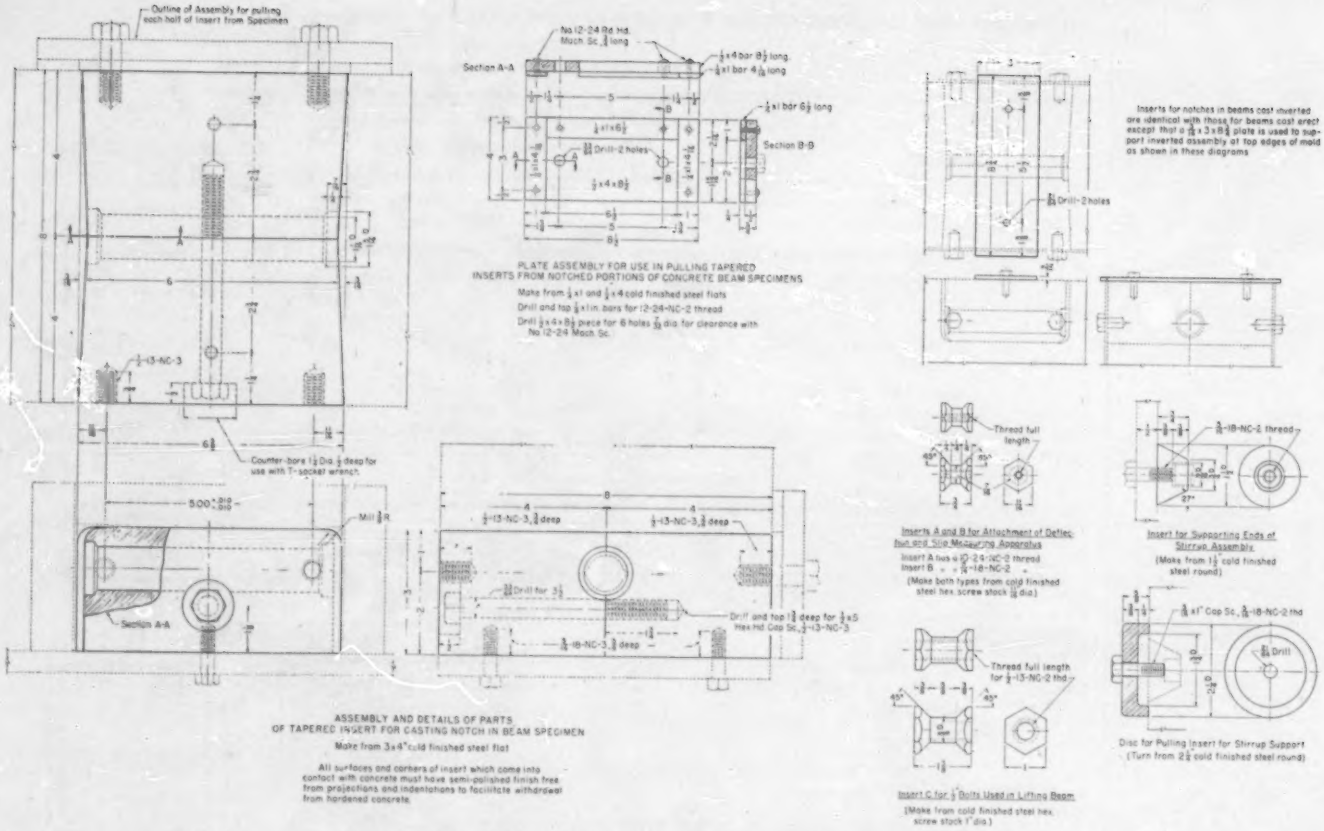


The above diagram illustrates marking of mold parts for Type C specimens. The first letter indicates the type of beam and the second letter the location of part in diagram of mold assembly. These marks are stamped into outer face of each part with steel letters ½ high. The 6 types of specimens (one complete mold for each type) are indicated in the table of dimensions above.

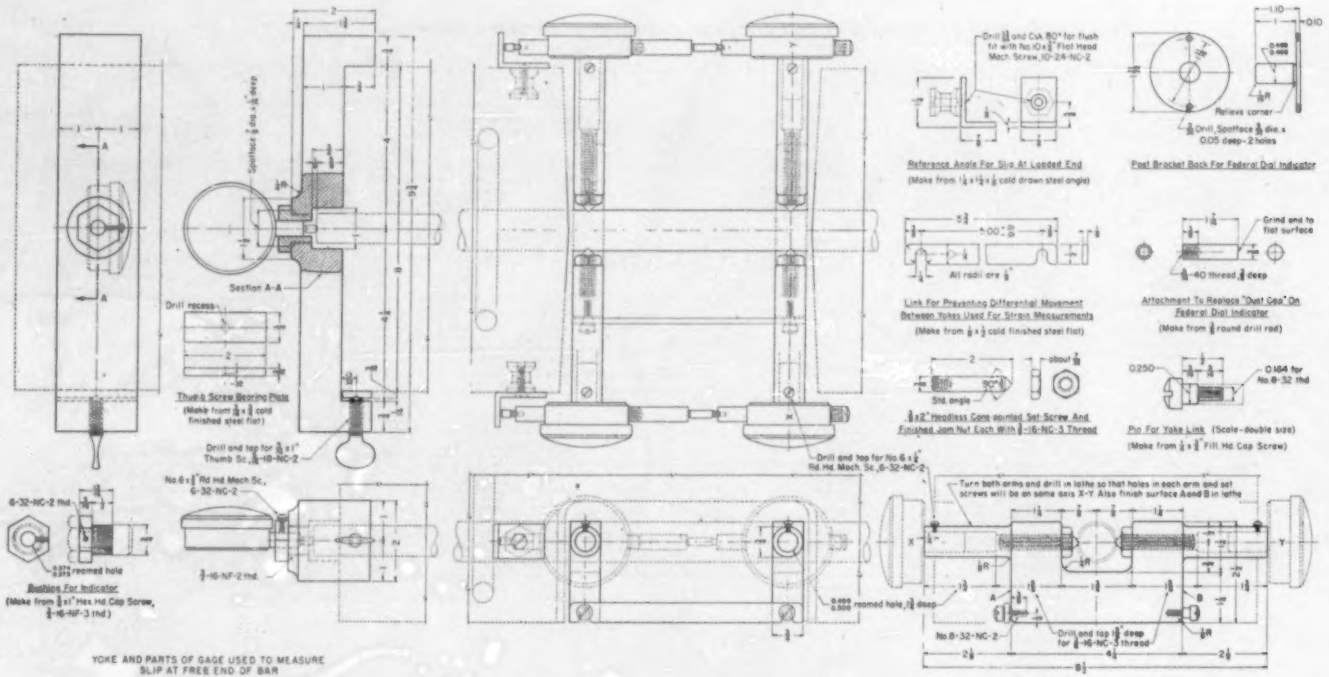
Cast-in Marks on Beam Specimens to Assist in Mounting Apparatus For Test

The pair of short dash lines denoted within circled areas of the side plates represent the location and length of V-shaped grooves milled into the inner face of width front and back plate. These grooves leave small projecting lines on the surface of the concrete and mark the correct position of bearing plates at load and reaction points during test of beam. Grooves are also milled into the inner face of all end plates to mark position of poke of gage for slip measurements on free end of bar.

Dwg. 5—Details of exterior parts of mold for beam specimens cast erect and inverted


Outline of Assembly for pulling each half of insert from Specimen
Section A-A
Counter-bore 1½ Dia. ½ deep for use with T-socket wrench
Mill ⅛ R
No 12-24 Rd Hd Mach Sc. ⅜ long
¼ x 4 bar 8½ long
¼ x 1 bar 4⅛ long
¼ x 1 bar 6½ long
Section B-B
Drill-2 holes
PLATE ASSEMBLY FOR USE IN PULLING TAPERED INSERTS FROM NOTCHED PORTIONS OF CONCRETE BEAM SPECIMENS
Make from ¼ x 1 and ¼ x 4 cold finished steel flats
Drill and tap ¼ x 1 in. bars for 12-24-NC-2 thread
Drill ¼ x 4 x 8½ piece for 6 holes ⅞ dia. for clearance with No 12-24 Mach Sc.
Drill for ⅜
Drill and tap ⅜ deep for ¼ x 5 Hex Hd Cap Sc. ¼-13-NC-3
⅜-16-NC-3 ⅜ deep
ASSEMBLY AND DETAILS OF PARTS OF TAPERED INSERT FOR CASTING NOTCH IN BEAM SPECIMEN
Make from 3 x 4" cold finished steel flat
All surfaces and corners of insert which come into contact with concrete must have semi-polished finish free from projections and indentations to facilitate withdrawal from hardened concrete.
Inserts for notches in beams cast inverted are identical with those for beams cast erect except that a ⅛ x 2 x 8½ plate is used to support inverted assembly at top edges of mold as shown in these diagrams
Drill-2 holes
Thread full length
Inserts A and B for Attachment of Deflection and Slip Measuring Apparatus
Insert A has a 10-24-NC-2 thread
Insert B " " ⅜-16-NC-2 "
(Make both types from cold finished steel hex screw stock ⅝ dia.)
⅜-16-NC-2 thread
Insert for Supporting Ends of Stirrup Assembly
(Make from 1½ cold finished steel round)
Thread full length for ½-13-NC-2 thd
Insert C for ½ Bolts Used in Lifting Beam
(Make from cold finished steel hex screw stock 1" dia.)
⅜ x 1" Cap Sc. ⅜-16-NC-2 thd
Drill
Disc for Pulling Insert for Stirrup Support
(Turn from 2½ cold finished steel round)

Dwg. 6—Assembly and details of inserts at interior of beam mold


Section A-A
Drill recess
Thumb Screw Bearing Plate
(Make from ¼ x ¾ cold finished steel flat)
Drill and tap for ⅜ x 1" Thumb Sc. ⅜-16-NC-2
No.6 x ¾" Rd Hd Mach Sc., 6-32-NC-2
6-32-NC-2 thd
⅜-16-NF-2 thd.
Bushing For Indicator
(Make from ⅜ x 1" Hex. Hd. Cap Screw, ⅜-16-NF-3 thd.)
Drill and tap for No.6 x ¾" Rd. Hd. Mach. Sc.,6-32-NC-2
No.8-32-NC-2
Drill and tap ⅜ deep for ⅜-16-NC-3 thread
Turn both arms and drill in lathe so that holes in each arm and set screws will be on same axis X-Y. Also finish surface A and B in lathe
Reference Angle For Slip At Loaded End
(Make from 1½ x 1½ x ⅛ cold drawn steel angle)
Post Bracket Back For Federal Dial Indicator
Relieve corner
Drill, Spotface ⅝ dia. x 0.05 deep-2 holes
All radii are ⅛"
Link For Preventing Differential Movement Between Yokes Used For Strain Measurements
(Make from ¼ x ½ cold finished steel flat)
Attachment To Replace "Dust Cap" On Federal Dial Indicator
(Make from ½ round drill rod)
Grind end to flat surface
⅝-40 thread ⅜ deep
Std. angle
⅜ x 2" Headless Cone pointed Set Screw And Finished Jam Nut Each With ⅜-16-NC-3 Thread
Pin For Yoke Link (Scale-double size)
(Make from ¼ x ¾ Fill. Hd. Cap Screw)
0.164 for No.8-32 thd
0.250

YOKE AND PARTS OF GAGE USED TO MEASURE SLIP AT FREE END OF BAR

(Make yoke from 2" square straight draw aluminum bar, alloy 17 S T.)

YOKES AND PARTS FOR GAGE USED TO MEASURE ELONGATION (PULL) AND SLIP AT LOADED END OF BAR

Only one link is to be used between each pair of yokes but link pins are to be provided on both shoulders of each yoke so that link can be inserted at either side of notch. The yokes in each pair are identical.

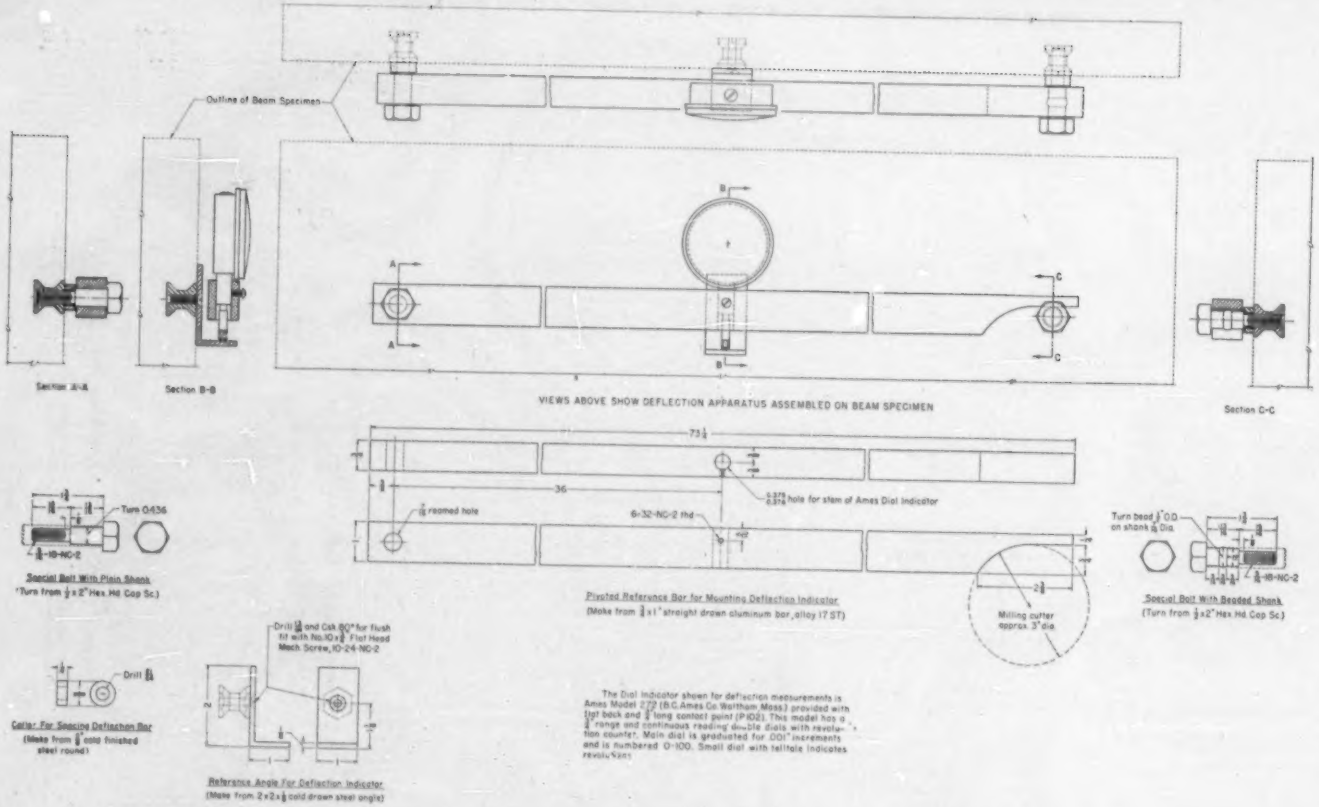
Dial indicators shown for slip and elongation measurements are Federal Model 0-2i (Federal Products Corp. Providence, R.I.). They are to have continuous reading double dials with revolution counter and a range of .2000". Main dial is graduated for .001" increments and is numbered 0-100. Small dial with hairline indicates revolutions.

Indicators for slip at free end of bar have flat backs and ¼ long contact points. Indicators for slip at loaded end have post bracket backs, ¼ long contact points and an attachment replacing usual "dust cap" indicators for elongation measurements have post bracket backs and points ⅛ long.

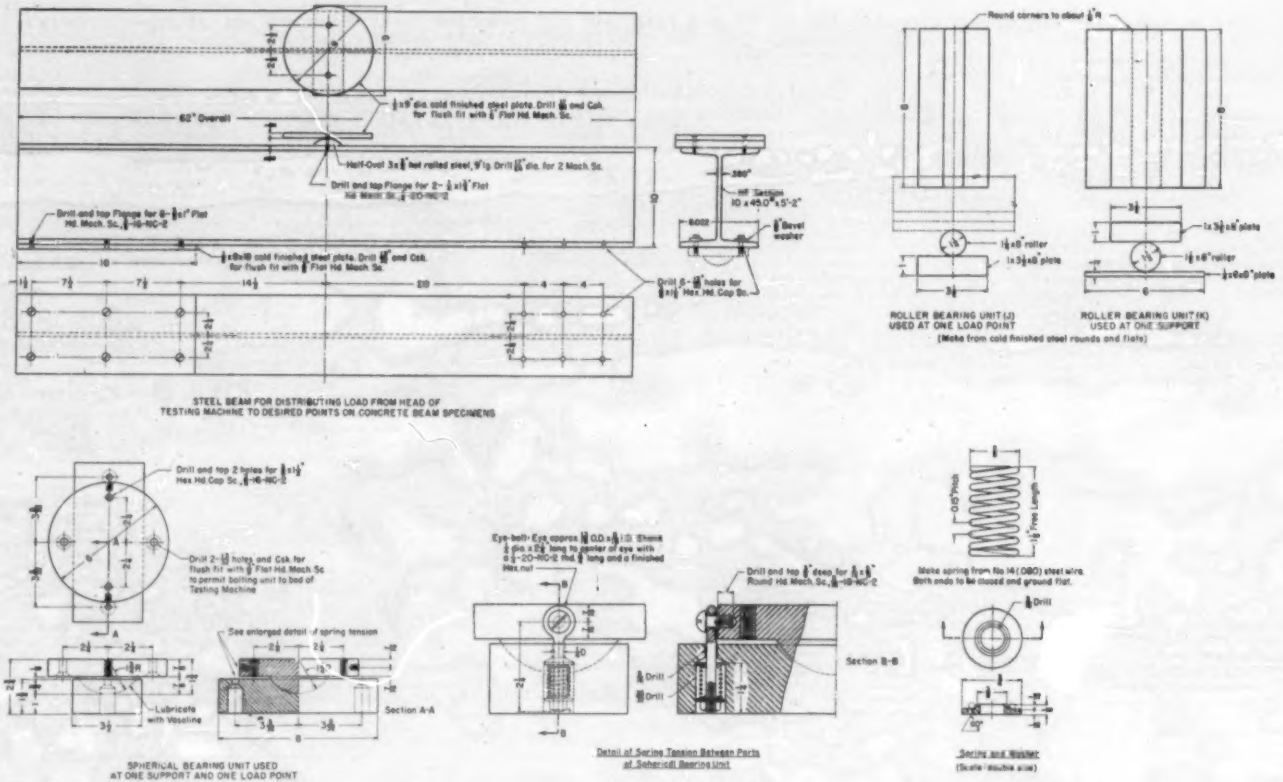
Extra contact points ⅛, ¼, ½ and ¾ long (two for each length) should be available. All points are of the regular type with ⅜ dia. and ⅛ contact rad.

Indicators should be checked and used over ranges of greatest accuracy, parts...only those for elongation measurements. At least three indicators, one with flat back and two with post backs, should be available as spares.

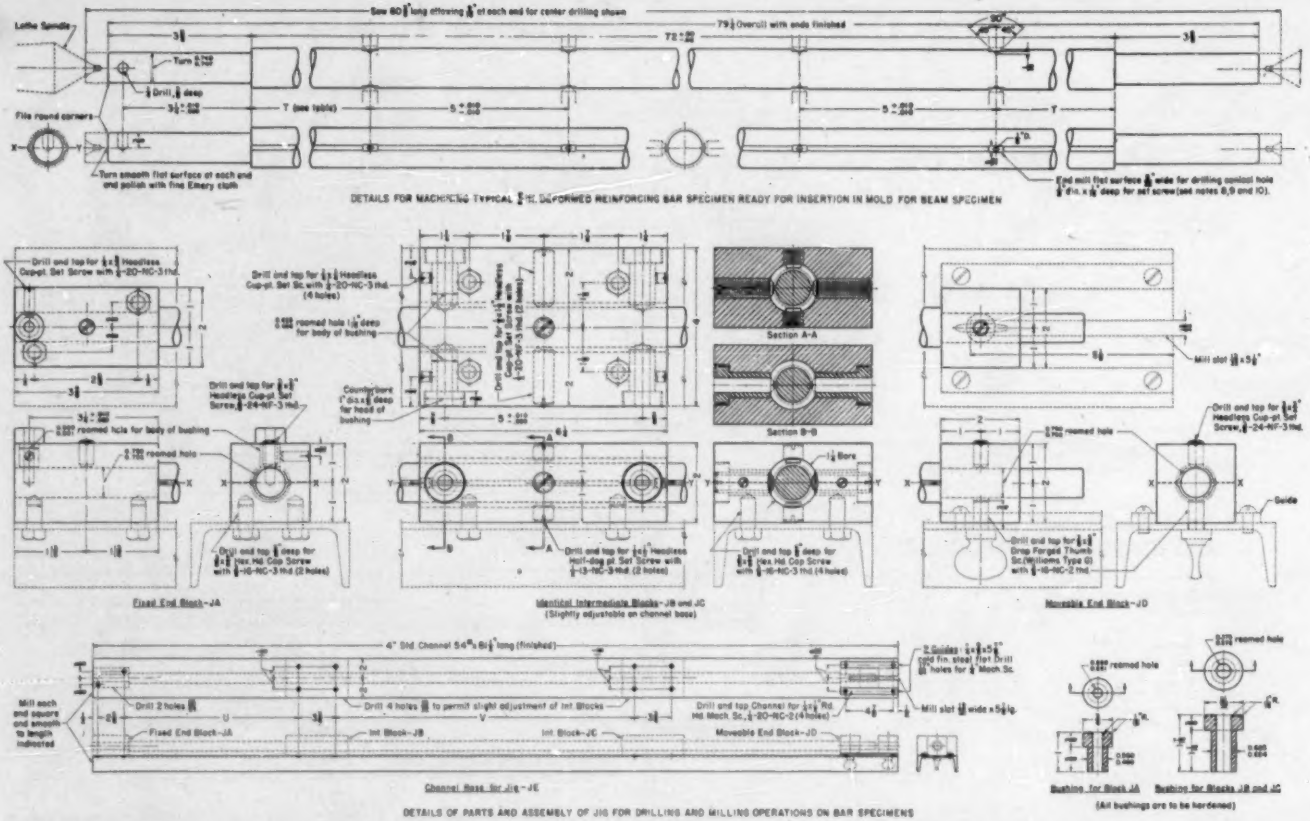
Dwg. 7—Assembly and parts of apparatus for measuring elongation and slip of bar


Outline of Beam Specimen
Section A-A
Section B-B
VIEWS ABOVE SHOW DEFLECTION APPARATUS ASSEMBLED ON BEAM SPECIMEN
Section C-C
Turn 0.436
Special Bolt With Plain Shank
(Turn from ½ x 2" Hex. Hd. Cap Sc.)
Drill
Collar For Spacing Deflection Bar
(Make from ⅜ cold finished steel round)
Drill ⅛ and Csk 80° for flush fit with No.10 x ½ Flat Head Mach. Screw, 10-24-NC-2
Reference Angle For Deflection Indicator
(Make from 2 x 2 x ⅛ cold drawn steel angle)
73½
36
⅛ reamed hole
6-32-NC-2 thd
0.375 / 0.374 hole for stem of Ames Dial Indicator
Pivoted Reference Bar for Mounting Deflection Indicator
(Make from ⅜ x 1" straight drawn aluminum bar, alloy 17 ST)
Milling cutter approx. 3" dia.
Turn bead ½" O.D. on shank ⅜ Dia.
Special Bolt With Beaded Shank
(Turn from ½ x 2" Hex. Hd. Cap Sc.)
The Dial Indicator shown for deflection measurements is Ames Model 2.72 (B.C. Ames Co. Waltham Mass.) provided with flat back and ½" long contact point (P102). This model has a ½" range and continuous reading double dials with revolution counter. Main dial is graduated for .001" increments and is numbered 0-100. Small dial with telltale indicates revolutions.

Dwg. 8—Assembly and parts of apparatus for measuring deflection of beam


62" Overall
Half-Oval 3x1/2" hot rolled steel, 9" lg. Drill 13/16" dia. for 2 Mach. Sc.
Drill and tap Flange for 2- 1/2 x1 1/2" Flat Hd. Mach. Sc. 1/2-20-NC-2
Drill and tap Flange for 8- 3/4 x1" Flat Hd. Mach. Sc. 3/4-16-NC-2
1/2 x8x10 cold finished steel plate. Drill 13/16" and C'sk. for flush fit with 1/2" Flat Hd. Mach. Sc.
Drill 8- 13/16" holes for 3/4 x1 1/2" Hex. Hd. Cap Sc.
Bevel washer
ROLLER BEARING UNIT (J) USED AT ONE LOAD POINT
ROLLER BEARING UNIT (K) USED AT ONE SUPPORT
(Made from cold finished steel rounds and flats)
STEEL BEAM FOR DISTRIBUTING LOAD FROM HEAD OF TESTING MACHINE TO DESIRED POINTS ON CONCRETE BEAM SPECIMENS
Drill and tap 2 holes for 3/4 x1 1/2" Hex Hd. Cap Sc. 3/4-16-NC-2
Drill 2- 13/16 holes and C'sk for flush fit with 1/2" Flat Hd. Mach. Sc to permit bolting unit to bed of Testing Machine
Lubricate with Vaseline
See enlarged detail of spring tension
Section A-A
SPHERICAL BEARING UNIT USED AT ONE SUPPORT AND ONE LOAD POINT
(Make parts from cold finished steel. Grind Spherical Bearing Surfaces to perfect fit.)
Eye-bolt-Eye approx 1/4 O.D. x 1/2 I.D. Shank 1/2 dia. x 2 1/2 long to center of eye with 1/2-20-NC-2 thd. 1 1/2 long and a finished Hex. nut
Drill and tap 1/2" deep for 1/2 x1 1/2" Round Hd. Mach. Sc. 1/2-13-NC-2
Section B-B
Drill
Detail of Spring Tension Between Parts of Spherical Bearing Unit
Make spring from No. 14 (.080) steel wire. Both ends to be closed and ground flat.
Spring and Washer
(Scale - double size)

Dwg. 9—Details of apparatus for supporting and applying load to beam specimens


Lathe Spindle
File round corners
Turn smooth flat surface at each end and polish with fine Emery cloth
T (see table)
End mill flat surface wide for drilling conical hole dia. x deep for set screw (see notes 8,9 and 10).
DETAILS FOR MACHINING TYPICAL DEFORMED REINFORCING BAR SPECIMEN READY FOR INSERTION IN MOLD FOR BEAM SPECIMEN
Section A-A
Section B-B
Mill slot x
Guide
Fixed End Block-JA
Identical Intermediate Blocks-JB and JC
(Slightly adjustable on channel base)
Movable End Block-JD
Mill each end square and smooth to length indicated
Drill 2 holes
Drill 4 holes to permit slight adjustment of Int Blocks
Fixed End Block-JA
Int Block-JB
Int Block-JC
Movable End Block-JD
Mill slot wide x
Channel Base for Jig-JE
DETAILS OF PARTS AND ASSEMBLY OF JIG FOR DRILLING AND MILLING OPERATIONS ON BAR SPECIMENS
Bushing for Block JA
Bushing for Blocks JB and JC
(All bushings are to be hardened)

Dwg. 10—Details for machining ⅞-in. deformed bar specimens and jig for use in drilling and milling operations on bar

TABLE AND NOTES ACCOMPANYING DRAWING 10

VALUES OF DIMENSION *T* FOR BAR AND *U*, *V* FOR BAR JIG

Specimen	Casting position	Length of embedment, in.	*T*	*U*	*V*
A	Erect	8	8.50	9.62	46.25
AA	Inverted	8	8.50	9.62	46.25
B	Erect	12	12.50	13.62	38.25
BB	Inverted	12	12.50	13.62	38.25
C	Erect	16	16.50	17.62	30.25
CC	Inverted	16	16.50	17.62	30.25

Reference marks on jig blocks

Arrows designated *X* and *Y* point to ends of blocks in jig which are to be marked by V-grooves, shown as dash lines, to indicate center line of holes for reference in mounting bar in jig as described in Notes 5 and 6 below. V-grooves should be about 0.02 in. wide and milled with same cutter as for V-groove detailed in DRAWING 5.

Notes on machining of bar

Proper machining of the deformed bar specimens under investigation constitutes one of the most important steps in carrying out the test program. The following notes indicate the sequence of machining operations and the use of the jig for drilling and milling the bar. These operations should not be conducted as a "production" job but as a "precision" job.

1. Center drill for lathe spindle each end of deformed bar specimen as nearly as possible on its longitudinal axis. If necessary, grind off lugs and seams for short distance at each end to leave round core which can be centered in chuck for center drilling.

2. Turn ends of bar to nominal ¾-in. diameter on lathe centers. It is very important to obtain *square shoulders* at *exact spacing* for proper location in jig for drilling and milling operations.

3. Turn ends of bar flat and to length indicated. Round corners with file and polish ends with fine emery cloth.

4. Install bar in jig by passing one end through blocks *JC* and *JB* into the hole of fixed end block *JA* and slipping movable end block *JD* over opposite end of bar. Block *JD* is clamped firmly in position by thumb screw when the turned shoulder at each end of bar is in intimate contact with the inner face of the adjacent end block.

5. Rotate bar in its end bearings until the parting line of the upper and lower halves of the bar (as rolled) coincides as nearly as possible with the horizontal center line *XX* of hole in each end block. (Center line is marked by V-grooves milled into inner face of each block). Cup-point set screws at the top of end blocks are then tightened to prevent rotation of bar.

6. Half-dog point set screws at the top and bottom of intermediate blocks *JB* and *JC* are now adjusted so that the parting line of bar coincides with center-line marks *YY* at ends of these blocks. This adjustment is very important. It provides for alignment of bars which are not perfectly straight and controls the accuracy of milling and drilling of holes which receive the points of set screws on yokes for elongation measurements. Cup-point set screws on opposite sides of each intermediate block hold the bar firmly within the center of the hole against lateral movements during milling and drilling.

7. When the bar is accurately positioned and firmly held at various points of the jig the ¾-in. diameter hole ⅜ in. deep at the extreme left end of bar is drilled. This hole is very important as it engages the special set screw in end plate of mold, as detailed in DRAWING 4, to control the position of the bar in the mold.

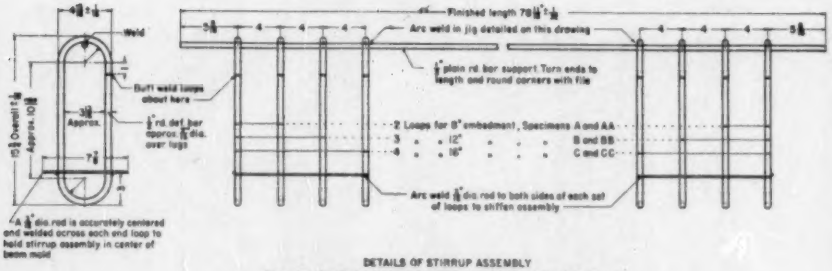
8. Through each bushing in blocks *JB* and *JC* mill a flat surface just deep enough to provide a flat area 5/32 in. wide for drilling gage points on parting line of bar. A Putnam No. A-78 high speed end mill ⅝ in. in diameter by 3¼ in. long with four flutes is contemplated for this operation. Keep cutting edges sharp to reduce chatter and vibration to a minimum.

9. Through the bushings drill conical holes ⅛ in. in diameter and 1/16 in. deep into flat milled surfaces of bar. These holes are to furnish firm bearing for 90-deg cone-pointed set screws in yokes of gages for measuring elongation and slip at loaded end of bar as shown in DRAWING 7. In drilling these holes use specially ground ⅛-in. high-speed Morse Twist Drill Co. drills provided with cutting edges having a lip angle of 45 deg (instead of the usual 59 deg) and meeting in a point as sharp as possible consistent with good cutting action.

10. All milling and drilling operations should be so conducted as to reduce the cross-sectional area of bar as little as possible consistent with achieving precision in locating firm bearings for set screws of yokes at gage points. Obviously, once a given bar specimen has been positioned in the jig it must not be moved until all drilling operations are completed. Only straight bar specimens should be selected for these tests.

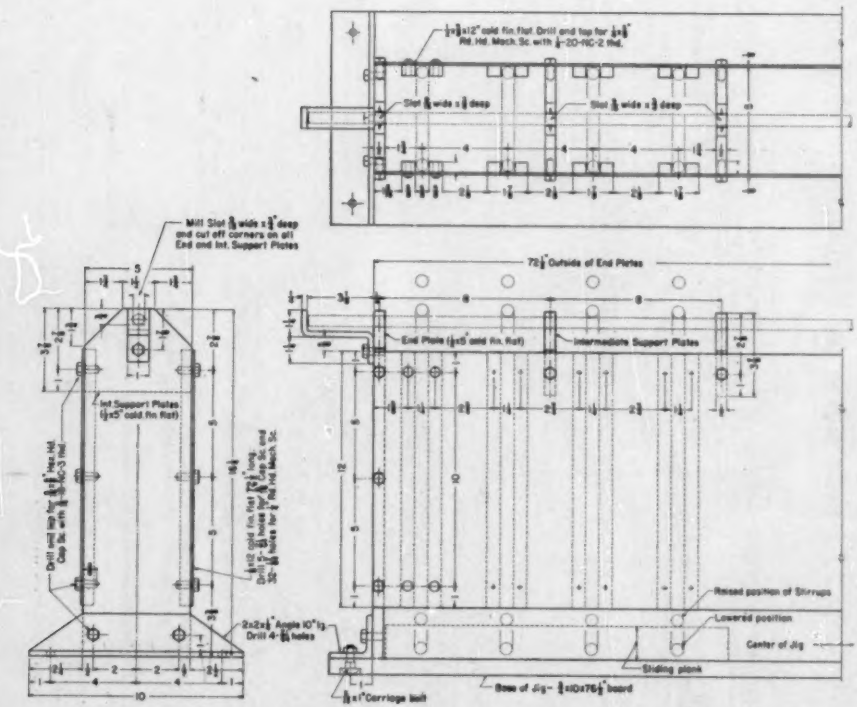
Rubber sleeves

These are approximately 1 in. O.D., ¾ in. I.D., and 3 in. long to cover the turned portion of bar between shoulder and end plate as detailed in DRAWING 4. The internal diameter should be small enough to provide a snug fit with bar and the ends should be cut square to a length of 3 in. ± 1/32.


Finished length 78⅛"±⅛
Arc weld in jig detailed on this drawing
Weld
Butt weld loops about here
Approx.
Approx. 10⅛
Overall 15⅛
7⅛
⅜" rd. def. bar approx. 6⅛ dia. over lugs
⅜" plain rd. bar support. Turn ends to length and round corners with file
2 Loops for 8" embedment, Specimens A and AA
3 " " 12" " " B and BB
4 " " 16" " " C and CC
Arc weld ⅜" dia. rod to both sides of each set of loops to stiffen assembly
A ⅜" dia. rod is accurately centered and welded across each end loop to hold stirrup assembly in center of beam mold

DETAILS OF STIRRUP ASSEMBLY

For a given length of embedment the stirrup assembly is identical whether the beam specimens are cast erect or inverted. The number of loops at each end varies with the embedded length of the deformed bar under test as indicated above.


Slot ⅝ wide x ½ deep
Slot ⅝ wide x ½ deep
Mill Slot ⅝ wide x ½" deep and cut off corners on all End and Int. Support Plates
Int. Support Plates (½x5" cold fin flat)
End Plate (½x5" cold fin. flat)
Intermediate Support Plates
Raised position of Stirrup
Lowered position
Center of Jig
Sliding plank
2x2x¼" Angle 10" lg. Drill 4-⅝ holes
Base of Jig- ¾x10x76" board
⅝x7" Carriage bolt

JIG FOR HOLDING LOOPS DURING WELDING TO HORIZONTAL SUPPORTING BAR OF STIRRUP ASSEMBLY

Use a plank about 1½x5x46" resting on base of jig to support each set of loops until ½" bar support is in place between end brackets of jig. By sliding each plank toward center of jig the loops will drop into normal position in contact with bar ready for arc welding.

Dwg. 11—Details of stirrup assembly and jig for holding loops during welding to horizontal supporting bar

Recommended Practice for Design of Concrete Pavements (ACI 325-58)*

Reported by ACI Committee 325

E. A. FINNEY
Chairman

HENRY AARON
LEO M. ARMS
JOHN A. BISHOP
FRANK B. BROWN
BENGT F. FRIBERG
A. T. GOLDBECK
ROBERT HORONJEFF
J. D. LINDSAY
JOSEPH H. MOORE
L. A. PALMER
G. S. PAXSON
THOMAS B. PRINGLE
L. W. TELLER
WILLIAM VAN BREEMEN
KENNETH B. WOODS
F. N. WRAY

SYNOPSIS

Recommendations are presented for the design of rigid concrete pavements and bases based on practice proved successful in the United States. It offers comprehensive directions to design rigid airport and highway pavements or bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. It includes recommendations for soil foundations, selection of slab dimensions, joints, and details, for reinforced or nonreinforced pavement. Recommendations for design of cement-stabilized bases, continuously reinforced pavement, prestressed pavement, and rigid resurfacings are not included since their use has not yet developed a generally applicable practice. Design practices of all principal state and federal agencies concerned with paving have been reviewed and related within the limits recommended.

CHAPTER 1—INTRODUCTION

100—General notation

A_s = area of steel required per ft width of pavement or joints, sq in.
E_c = modulus of elasticity of concrete, psi
E_s = modulus of elasticity of steel, psi
F = coefficient of friction between slab and subgrade
f_b = allowable concrete bearing stress, psi
f_b' = ultimate bearing stress of concrete, psi
f_c' = ultimate compressive strength of concrete, psi
f_s = allowable tensile stress in steel, psi
f_t' = ultimate tensile strength of concrete at time of cracking, psi

*Adopted as a standard of the American Concrete Institute at its 54th annual convention, Feb. 25, 1958, as reported by Committee 325; ratified by letter ballot June 1, 1958. Title No. 55-2 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Separate prints in covers are available at 50 cents each to ACI members ($1.00 to nonmembers). **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

MR = modulus of rupture, psi
f_t = allowable tensile stress in concrete, psi
FS = factor of safety
G = modulus of support of the concrete mass, lb per cu in.
h = pavement thickness, in.
I = moment of inertia, in.^{4}
k = modulus of foundation support to slab, lb per cu in.
L = length of slab between joints, ft
L' = distance between the joint in question and the nearest free joint or edge, ft
p = subgrade support reaction under the slab, psi
P = wheel load, lb
a = radius of a circular area equal to that of the actual area of contact of a single tire with pavement, in. For dual tires, a is the radius of a circle of area equal to the sum of the areas of contact of the two tires plus the area between them, assuming that the contact area of each is circular.
l = radius of relative stiffness, in.
u = allowable bond stress between concrete and steel, psi
w = weight of the pavement slab, psf
μ = Poisson's ratio (for concrete assume $\mu = 0.15$)
Z = slab deflection, positive downward, in.
a_1 = $a\sqrt{2}$

101—General scope

This report presents recommended practice for the structural design of concrete pavements for highways and airports. It includes recommendations for subgrades and subbases in relation to their effect on the structural design of the pavement. Recommendations are presented for the design of slab dimensions, for the design of joints and joint details, and for the design of steel reinforcement. The report is intended for use in conjunction with "Specifications for Concrete Pavements and Concrete Bases" (ACI 617). It covers both nonreinforced and reinforced pavements but offers no recommendations for continuously reinforced pavements nor for concrete resurfacings.

102—General definitions

Concrete—A mixture of cement, fine aggregate, coarse aggregate, and water, which may include an approved admixture.

Concrete pavement—A rigid slab or series of slabs of portland cement type concrete, cast on a native soil subgrade or on a prepared subbase, generally exposed to weather, and designed to support traffic loads.

Crack—A permanent fissure or open seam within a concrete pavement at which tensile stress in the concrete has exceeded the tensile strength of the concrete.

Deformed bar—A reinforcing bar conforming to "Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement" (ASTM A 305). Bars not conforming to these specifications should be classed as plain bars.

Joint—The constructed junction of adjacent slabs interrupting the continuity of the pavement.

Reinforcement—Steel embedded in a concrete pavement to insure intimate contact of adjacent slab faces at cracks.

Slab—For design purposes, a monolithic portion of concrete pavement,

bounded by joints or edges, within which continuity of tensile stress in the concrete is possible.

Slab length—Slab length in an uncracked pavement is the distance between adjacent transverse joints of any kind.

Subbase—Subbase, when used under a rigid pavement, refers to a relatively thin layer of selected or specified materials of a prescribed thickness placed between the subgrade and rigid pavement for such purposes as improvement of drainage, reduction in frost damage, prevention of pavement pumping, or moisture control of high volume change subgrade soils.*

Welded wire fabric—A two-way reinforcement system, fabricated from cold-drawn steel wire, having parallel longitudinal wires welded at regular intervals to parallel transverse wires and conforming to "Specifications for Welded Steel Wire Fabric for Concrete Reinforcement" (ASTM A 185 and AASHO M 55).

103—Reference specifications cited in this recommendation

(a) The standard of the American Concrete Institute listed below (with serial designation including the year of latest revision) is declared to be a part of this recommendation, the same as if fully set forth elsewhere herein: "Specifications for Concrete Pavements and Concrete Bases" (ACI 617).

(b) Further references which discuss the purposes and intentions of this recommendation and which set forth research data and theoretical considerations and derivations justifying this recommendation are the following separately published reports of the four subcommittees of Committee 325:

1. Subcommittee I, ACI Committee 325, "Considerations for Construction of Subgrades and Subbases for Rigid Pavement," ACI JOURNAL, Aug. 1956, *Proc.* V. 53, pp. 145-155.

2. Subcommittee II, ACI Committee 325, "Considerations in the Selection of Slab Dimensions," ACI JOURNAL, Nov. 1956, *Proc.* V. 53, pp. 433-454.

3. Subcommittee III, ACI Committee 325, "Structural Design Considerations for Pavement Joints," ACI JOURNAL, July 1956, *Proc.* V. 53, pp. 1-28.

4. Subcommittee IV, ACI Committee 325, "Design Considerations for Concrete Pavement Reinforcement for Crack Control," ACI JOURNAL, Oct. 1956, *Proc.* V. 53, pp. 337-362.

104—Materials and tests

All materials to be used in constructing the subgrade, subbase, and the concrete pavement should conform to the requirements of "Specifications for Concrete Pavements and Concrete Bases" (ACI 617).

CHAPTER 2—CONCRETE PROPERTIES AND ALLOWABLE STRESSES

201—Concrete properties

Concrete properties as specified in "Specifications for Concrete Pavements and Concrete Bases" (ACI 617) should be the minimum required.

*This definition of subbase, for purposes of pavement design, includes both subbases and soil-aggregate base courses as defined for specification purposes in "Specifications for Concrete Pavements and Concrete Bases" (ACI 617).

202—Allowable unit stresses in concrete

(a) The unit stresses in pounds per square inch in concrete to be used in design of concrete pavements should not exceed the following values:

$f_t = 0.50$ MR

$u = 0.10 f_c'$, but not more than 350 psi for deformed bars or welded wire fabric

$u = 0.045 f_c'$, but not more than 160 psi for plain bars

(b) The unit bearing stresses in pounds per square inch in concrete under dowels to be used in design of concrete should not exceed the values in Table 202(b).

TABLE 202(b)—ALLOWABLE UNIT BEARING STRESSES IN CONCRETE UNDER DOWELS

Dowel diameter, in.	f_b, psi
¾	3200
⅞	3100
1	3000
1⅛	2900
1¼	2800
1⅜	2650
1½	2500
2	2000

203—Allowable unit stresses in steel

The unit stresses in pounds per square inch in steel for use in concrete pavements should not exceed the following values:

Type and grade of steel	Minimum yield point, psi	f_s, psi
Billet and axle steel, structural grade	33,000	22,000
Billet and axle steel, intermediate grade	40,000	27,000
Rail steel or billet and axle steel, hard grade	50,000	33,000
Cold drawn wire	56,000	37,000

CHAPTER 3—SUBGRADES AND SUBBASES

301—Definitions

Drainage—Drainage is the control of water accumulations on or in subgrades or subbases as necessary to insure satisfactory performance of the pavement.

Filter course—A subbase graded so that underlying fine-grained subgrade soils are prevented from penetrating any appreciable distance upward into this layer thus preventing detrimental subgrade pumping.

Frost action—Frost action in soil is a phenomenon involving freezing, thawing, and movement of moisture in soil, and consequent changes in its physical properties.

Frost heave—Frost heave is the surficial expression of the volume changes which take place when (1) soil water is frozen and (2) ice lenses form within the zones of freezing.

Frost-susceptible soil—Material possessing sufficient permeability and a relatively high degree of capillarity such as silts and varved clays.

Pumping—Pumping in a concrete pavement is a process in which the subgrade soil and water form a suspension which is forced to the surface at the edges, cracks, and joints of the pavement under traffic loads.

Shrinkage and swelling—These phenomena represent one portion of the general category of volume change in soils and are caused by varying soil moisture contents.

302—Pavement pumping

Pumping occurs at or near joints and cracks and along the edges of concrete pavement, and may be recognized by soil stains on the pavement and soil-water slurries at pavement edges. Three factors are essential to pumping: (1) free water must be present beneath the slab; (2) axle loadings must be heavy and frequent; and (3) the subgrade soil must be capable of going into suspension. If any one of the foregoing factors is absent, pumping will not occur. Breaking of pavements will result from continued pumping.

The use of granular materials as relatively thin subbases, when designed as a filter course, the use of selected soils and granular materials as thicker subbases, or selected soils and granular materials stabilized with a cementing agent are effective methods of preventing pumping on new construction. Pumping may also be minimized through the use of adequately designed load-transfer devices which reduce pavement deflection and prevent faulting, and through the use of adequate reinforcement.

303—Subbases

(a) *Subbase functions*

1. The subbase for a rigid pavement is a compacted layer of planned thickness that is placed immediately under the pavement and on the subgrade to serve one or all of the following functions:

a. To provide uniform, stable, and permanent support for the pavement.
b. To prevent or reduce the damaging effects of frost action.
c. To improve drainage and, when properly drained, minimize the accumulation of water under the pavement.
d. To prevent pumping action at joints, cracks, and edges of pavement.
e. To reduce volume changes in the subgrade and to minimize the detrimental effects of such volume changes on the pavement.

2. Thicknesses greater than 6 in. are used to replace frost-susceptible subgrade soils, minimize the effect of volume change, and to provide increased subbase-pavement strength, in addition to preventing pumping.

(b) *Minimum load-supporting value*—Where economical, the finished subbase should have a load-supporting value such that when it is loaded in increments with a 30-in. diameter steel plate, it will develop a modulus of foundation reaction, k, of not less than 200 lb per cu in. The subgrade modulus, k, should be evaluated by loading tests made directly on the finished subbase and/or trial sections of the subbase constructed in both cut and fill areas and at the transition between cut and fill areas.

(c) *Requirements to prevent pumping*—Where the subgrade soil is capable of going into suspension, filter course subbases 3 to 6 in. thick will prevent pumping.

304—Effect of subbase and subgrade on pavement thickness design

The thickness of pavement required for given traffic loads and other conditions is not affected significantly by the presence or absence of a granular subbase layer of the usual thicknesses, but strong stabilized subbases may have an effect. The supporting power of the subgrade or combined subgrade and subbase does affect the thickness of pavement required, particularly where heavy wheel or axle loads are involved.

(a) *Corps of Engineers method*

1. Under normal conditions, or where freezing is moderate, or where the water table is deep, a 4 in. minimum thickness of subbase is used.

2. When frost action is anticipated, modifications in the design for normal conditions are made. With an F-4 subgrade soil, the worst from the standpoint of frost susceptibility, a total thickness of subbase and pavement equal to the depth of frost penetration is used. Subgrade soils of less severe frost susceptibility are associated with a design modification that allows for a reduction in strength due to the frost action. A reduction in the modulus of subgrade reaction is made based upon appropriate charts. A thickness of subbase is used equal to the thickness of the slab designed assuming the reduced k value.

(b) *Navy method*—The thickness of subbase to be used is dependent upon the results of loading tests made on the trial sections during the evaluation of the subgrade.

(c) *Portland Cement Association method*—A subbase is added to the pavement when nonuniform soil conditions exist or when detrimental volume changes or pumping are anticipated. The thickness of subbase is arbitrary and is recommended as one-half to one-third of the depth of frost penetration, 4 to 12 in. in cases of high volume change soils, and 4 to 6 in. in the case of anticipated pumping action.

(d) *Civil Aeronautics Administration method*—A constant subgrade modulus is assumed as a result of the provision of variable subbase thicknesses which depend on wheel loading, subgrade soil, drainage conditions, and frost penetration.

CHAPTER 4—SLAB DIMENSIONS

400—Notations

y = longitudinal direction of pavement

x = transverse direction normal to y

$t(y), t(x)$ = tensile stress in slab, y and x directions, respectively

$\sigma(y), \sigma(x)$ = temperature warping stress, y and x directions, respectively

a_e = semi-major or semi-minor axis of tire contact area, but in same direction as x, in.

b_e = semi-major or semi-minor axis of tire contact area, but in same direction as y, in.

B,C,R,S = place and area coefficients

e = thermal coefficient of expansion for concrete

t = differential temperature, top to bottom of slab, deg F

401—Slab lengths

(a) *Nonreinforced concrete pavements*

1. The spacing of contraction cracks in nonreinforced concrete pavements without transverse joints other than construction joints placed at the end of each day of work may be predicted by the formula:

$$L = \frac{2f_t'^*}{F} \text{ in ft} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

2. Warping stresses may be assumed constant for 6-in. slabs longer than 18 ft and for 9-in. slabs longer than 24 ft. For longer slabs, maximum warping stresses in 9-in. slabs may be taken as 50 percent greater than those in 6-in. slabs. For shorter slabs, the warping stress is greater in 6-in. slabs. Increases in k values result in increased warping stresses in short slabs. See Fig. 401(a)1 and 401(a)2 by Kelley† for design data.‡

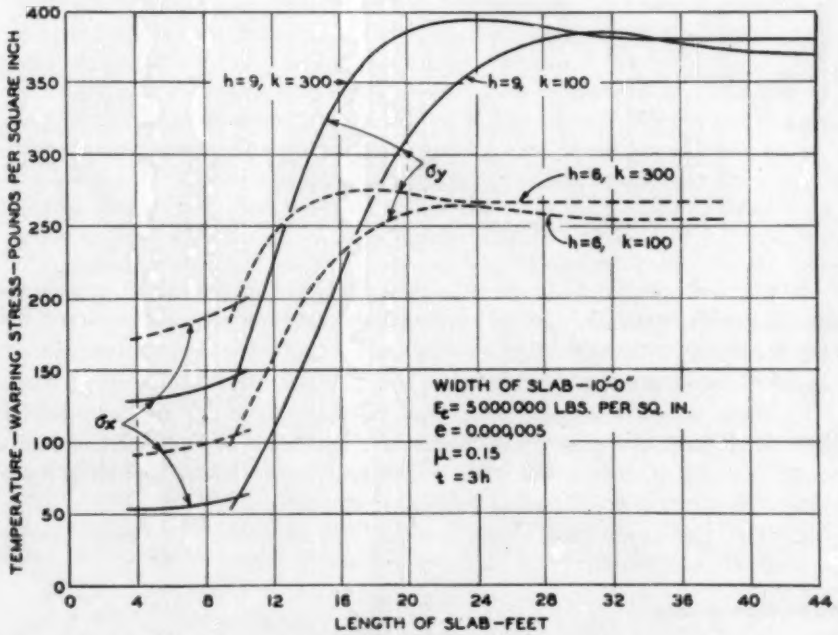


Fig. 401(a)1—Temperature-warping stresses—interior of slab

3. For nonreinforced concrete pavements, under average conditions a transverse joint spacing of 20 ft is recommended to prevent intermediate cracking. For unusual conditions this spacing may be varied to 15 or 25 ft.

*Unless the curing conditions prevent early shrinkage, f_t' will be very low, often based on an age less than 24 hr.

†Kelley, E. F., "Application of the Results of Research to the Structural Design of Concrete Pavements," ACI JOURNAL, June 1939, *Proc.* V. 35, pp. 437-464.

‡Warping stresses have never been measured directly. Theoretical predictions have neglected and strain measurements have included the effect of creep which reduces these stresses.

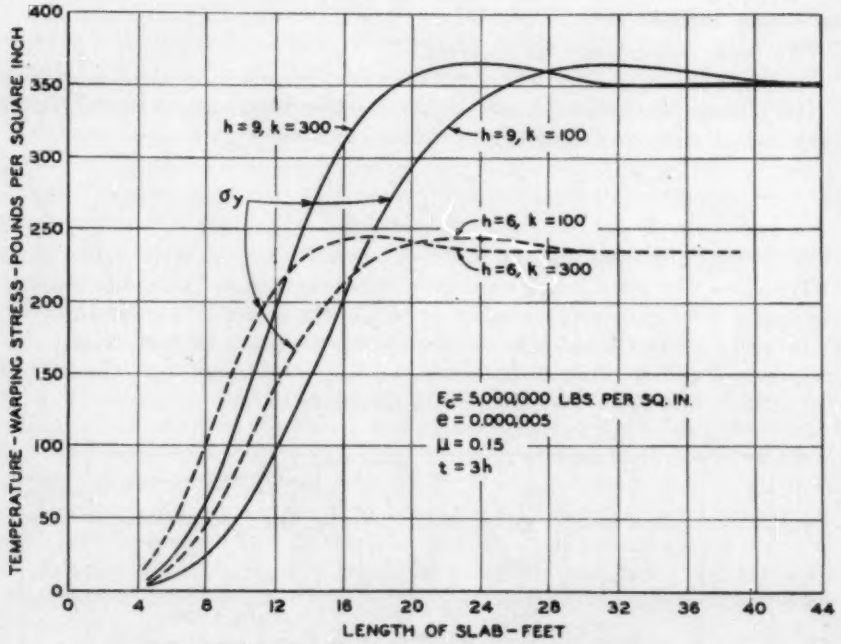


Fig. 401(a)2—Temperature-warping stresses—edge of slab

Where load transfer at joints is to depend wholly or partially upon aggregate interlock, expansion joints intermediate between structures except for special conditions should be eliminated.

(b) *Reinforced concrete pavements*

1. Since effective crack control can be maintained by the addition of distributed steel, the economical length of a reinforced concrete slab depends upon: (a) cost of distributed steel; (b) initial cost of joints, including load-transfer devices; and (c) cost of joint maintenance.

2. The commonly used joint spacing for reinforced pavement varies between 40 and 100 ft.

402—Slab widths

(a) Traffic requirements make a 12-ft minimum lane-width necessary for maximum capacity of mixed (passenger and commercial) traffic.

(b) For highway pavement, to prevent irregular longitudinal cracking, a maximum spacing of 12 ft between longitudinal joints is recommended.

(c) For airport paving, to prevent irregular longitudinal cracking, longitudinal construction joints should be spaced at not more than 25 ft. For pavements 10 in. or less in thickness, an intermediate weakened plane joint of the dummy type should be required.

403—Slab thickness

(a) *Basic concepts for design*

1. Warping stresses due to differential temperatures between the top and bottom of the slab are daily occurrences. Their magnitude is high and they, combined with load stresses, can and do produce transverse cracking in the pavement.

2. Wheel loads also produce high bending stress, especially along the edges of the slab and near the corners, but these are of lesser amount in the center of the slab. Excessive bending can produce accentuated pumping.

3. Load transfer across joints or cracks helps to relieve joint, edge, and corner stresses.

4. Distributed steel may not prevent cracking, but it does hold the slab together at the cracks so that faulting of the pavement at these cracks is avoided. When cracks occur, the high warping stress which formerly existed in the vicinity of the crack becomes negligible.

5. Subgrade friction in slabs longer than suitable for unreinforced pavements should be resisted by longitudinal steel, which should be sufficiently strong to prevent its failure due to excessive tension.

6. Concrete suffers fatigue under repeated heavy loading and the importance of this fact must at least be considered in any theory of pavement design. However, if the maximum stress is no greater than half the modulus of rupture, fatigue will not be a factor in design. It is recommended that the pavement thickness be so designed that under the assumed maximum wheel load the stress will be restricted to half the modulus of rupture.

7. Concrete usually fails by tension produced either directly or through bending.

8. When designing nonreinforced slabs, warping stresses as well as load stresses must be considered, and the joints should be spaced so as to prevent intermediate cracks from forming due to combined stresses from these two effects.

9. When designing reinforced concrete pavement, warping stresses need not be considered in addition to load stresses.

(b) *Design theory*

1. The accepted theory (developed by Westergaard) involves, among others, one important assumption that is expressed in the simple formula:

$$p = k Z \qquad (2)$$

Obviously, Eq. (2) is not applicable to nonuniform or spot support of the slab. The theory does not pertain to the case of unsupported corners and edges of the slab. The modulus of foundation support k is assumed to be constant over the entire paved area and during the life of the pavement.

2. For finding the modulus of foundation support, k, it is considered that a satisfactory procedure is by the use of loading tests with a 30-in. diameter steel plate. This test should be made, whenever possible, on trial sections of variable thicknesses of compacted subbase or of compacted subgrade if

the subgrade material is suitable for a subbase. The control of moisture and density of the subbase is of paramount importance in this connection.

3. Wheel loads produce flexural stresses in a concrete pavement. The pavement fails under combined loading and warping conditions which cause a flexural stress equal to or greater than its flexural strength or modulus of rupture. The factor of safety, FS, against overloading of the slab is:

$$FS = \frac{MR}{f_t} \quad \ldots\ldots (3)$$

In this expression, MR is the modulus of rupture (psi) and f_t is the maximum principal bending stress (psi). A minimum factor of safety of 2 should be provided, except for certain airport pavements where the safety factor may be less, depending on the design wheel load.

4. If the pavement is designed so that the maximum stress is no greater than half the modulus of rupture, fatigue will not enter into the design problem as an important factor. The pavement thickness should be so designed that under the assumed maximum wheel load the stress will be restricted to roughly half the modulus of rupture. The design load should be greater than the legal load limit and it is well to assume an additional 20 percent for impact.

5. Separate design criteria are applicable for concrete airport and highway pavements. For airport pavements, computations of wheel load stresses for any given load and for different thicknesses of concrete have reference to loading at an interior point of the slab. In the case of concrete highway pavements the computed stresses have reference to protected or unprotected corner slab loading [see Section 403(d)].

(c) *Design application for airport pavement**

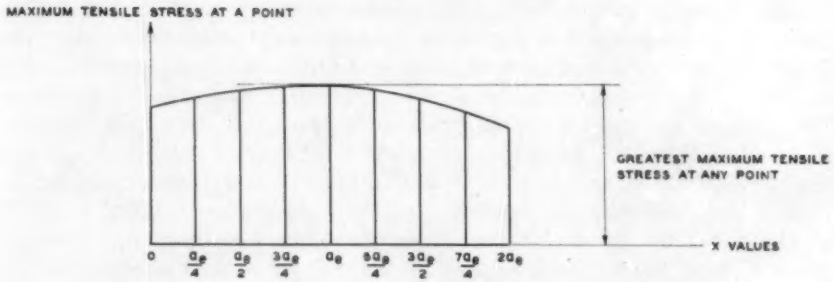
1. Single tire loading.

The choice of coordinate axes is arbitrary. Let y be in the longitudinal direction of the concrete highway or runway pavement and let the x direction be normal to the y direction in the plane of the pavement. Let the corresponding wheel load stresses be designated as $t(y)$ and $t(x)$. The tire contact area tends to be of elliptical shape, the major axis having a length of from 1.5 to 2.0 times the length of the minor axis. Let a_e and b_e denote the semi-axes of the ellipse, a_e being in the x direction and b_e in the y direction. If the elliptically shaped tire contact area is oriented with the long axis perpendicular to the longitudinal direction, then a_e will be the semi-major axis and b_e the semi-minor axis, respectively. Then the Westergaard expression for the concrete tensile stresses in the y and x directions at the bottom of the slab under the center of the tire contact area is as follows (Poisson's ratio assumed to be 0.15):

$$\begin{bmatrix} t(y) \\ t(x) \end{bmatrix} = \frac{P}{h^2}\left[0.316 \log_{10} \frac{E_c h^3}{k\left(\frac{a_e + b_e}{2}\right)^4} \pm (0.203)\frac{(a_e - b_e)}{(a_e + b_e)}\right] \quad \ldots\ldots (4)\dagger$$

*See Appendix A402.

†Westergaard, H. M., "New Formulas for Stresses in Concrete Pavements for Airfields," *Transactions*, ASCE, V. 113, 1948, pp. 425-444, and "Stress Concentrations in Plates Loaded over Small Areas," *Proceedings*, ASCE, V. 68, Apr. 1942, pp. 509-534.



TYPICAL GRAPH OF COMPUTED STRESSES

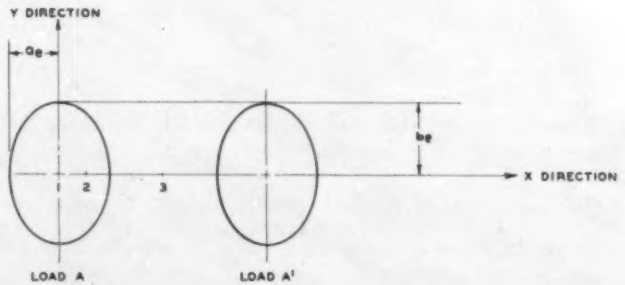


Fig. 403(c)—Dual tire loading on concrete pavement

In these expressions, the minus sign on the right corresponds to $t(x)$ and the plus sign to $t(y)$.

2. Dual tire loading

For dual tire loading, the following expressions are used, assuming that Poisson's ratio for concrete is 0.15:

$$\left.\begin{matrix} t(x) \\ t(y) \end{matrix}\right] = \frac{3P}{2\pi h^2}\left[(1.15)(B+R) \mp (0.425)(C+S)\right] \quad \ldots\ldots\ldots\ldots (5)^*$$

The coefficients B and C are "place" and R and S are "area" coefficients, and are determined numerically by the placing of loads and the shapes and sizes of the loaded areas. The total load is distributed uniformly over the tire contact area. If the x and y axes are axes of symmetry with respect to the load, then the place coefficients are $C = 0$ and $B = 0.1159$. For this symmetrical case Eq. (5) reduces to the following form:

$$\left.\begin{matrix} t(x) \\ t(y) \end{matrix}\right] = \frac{3P}{2\pi h^2}\left[(1.15)(0.1159+R) \mp (0.425S)\right] \quad \ldots\ldots\ldots\ldots (6)^\dagger$$

*See Appendix A402.
†Westergaard, H. M., *op. cit.*

With reference to Fig. 403(c), the stresses at points (such as *1* and *2*) in the concrete below Load A are *within* Ellipse A and *outside* Ellipse A'. The Point *3* is outside both ellipses. The expressions for the coefficients, R and S, are different for points inside and outside the ellipse. Since Eq. (6) is applicable only for loads which are symmetrical about the x and y axes, it is necessary to calculate R and S for Load A, also R and S for Load A', and then calculate the stress at any point, say Point *2*, due to Load A, and due to Load A' independently, and then by superposition the total stress at Point *2* can be obtained. For the stress at points within the ellipse such as *1* and *2* due to Load A alone, the following expressions are used for computing the numerical values of R and S:

$$\left.\begin{aligned} R &= 0.5000 + \log_e\left[\frac{2l}{a_e + b_e}\right] - \frac{x^2}{a_e^2 + a_e b_e} - \frac{y^2}{a_e b_e + b_e^2} \\ S &= \frac{b_e - a_e}{a_e + b_e} - \frac{2a_e b_e}{(a_e + b_e)^2}\left[\frac{x^2}{a_e^2} - \frac{y^2}{b_e^2}\right] \end{aligned}\right\} \quad \ldots\ldots\ldots (7)$$

For Point *1*, x and y are both zero since this point is the origin. For Point *2*, x has a finite value but y is zero since the point is on the x axis.

TABLE 403(d)—VALUES FOR *I* CORRESPONDING TO DIFFERENT VALUES OF *k* AND *h

k, lb per cu in.	50	100	150	200	250	300	350	400	500
h, in.	Radius of relative stiffness, *l*								
6	34.84	29.30	26.47	24.63	23.30	22.26	21.42	20.72	19.59
6.5	36.99	31.11	28.11	26.16	24.74	23.64	22.74	22.00	20.80
7	39.11	32.89	29.72	27.65	26.15	24.99	24.04	23.25	21.99
7.5	41.19	34.63	31.29	29.12	27.54	26.32	25.32	24.49	23.16
8	43.23	36.35	32.85	30.57	28.91	27.62	26.58	25.70	24.31
8.5	45.24	38.04	37.37	31.99	30.25	28.91	27.81	26.90	25.44
9	47.22	39.71	35.88	33.39	31.58	30.17	29.03	28.08	26.55
9.5	49.17	41.35	37.36	34.77	32.89	31.42	30.23	29.24	27.65
10	51.10	42.97	38.83	36.14	34.17	32.65	31.42	30.39	28.74
10.5	53.01	44.57	40.28	37.48	35.45	33.87	32.59	31.52	29.81
11	54.89	46.16	41.71	38.81	36.71	35.07	33.75	32.64	30.87
11.5	56.75	47.72	43.12	40.13	37.95	36.26	34.89	33.74	31.91
12	58.59	49.27	44.52	41.43	39.18	37.44	36.02	34.84	32.95
12.5	60.41	50.80	45.90	42.72	40.40	38.60	37.14	35.92	33.97
13	62.22	52.32	47.27	43.99	41.61	39.75	38.25	36.99	34.99
13.5	64.00	53.82	48.63	45.26	42.80	40.89	39.35	38.06	35.99
14	65.77	55.31	49.98	46.51	43.98	42.02	40.44	39.11	36.99
14.5	67.53	56.78	51.31	47.75	45.16	43.15	41.51	40.15	37.97
15	69.27	58.25	52.63	48.98	46.32	44.26	42.58	41.19	38.95
15.5	70.99	59.70	53.94	50.20	47.47	45.36	43.64	42.41	39.92
16	72.70	61.13	55.24	51.41	48.62	46.45	44.70	43.23	40.88
16.5	74.40	62.56	56.53	52.61	49.75	47.54	45.74	44.24	41.84
17	76.08	63.98	57.81	53.80	50.88	48.61	46.77	45.24	42.78
17.5	77.75	65.38	59.08	54.98	52.00	49.68	47.80	46.23	43.72
18	79.41	66.78	60.35	56.16	53.11	50.74	48.82	47.22	44.66
19	82.70	69.54	62.84	58.48	55.31	52.84	50.84	49.17	46.51
20	85.95	72.27	65.30	60.77	57.47	54.92	52.84	51.10	48.33
21	89.15	74.97	67.74	63.04	59.62	56.96	54.81	53.01	50.13
22	92.31	77.63	70.14	65.28	61.73	58.98	56.75	54.89	51.91
23	95.44	80.26	72.52	67.49	63.83	60.98	58.68	56.75	53.67
24	98.54	82.86	74.87	69.68	65.90	62.96	60.58	58.59	55.41

*E_c = 4,000,000 psi and μ = 0.15

Radius of relative stiffness = $l = \sqrt[4]{\frac{E_c(h^3)}{12(1-\mu^2)\,k}} = 24.162\sqrt[4]{\frac{h^3}{k}}$

The stresses in concrete at interior points of Ellipse A due to the Load A', the stresses at points such as 3 which is outside both ellipses, and the stresses within Ellipse A' due to Load A require for their computation the use of the following expressions for R and S:

$$\left.\begin{aligned} R &= 0.5000 + \log_e \frac{2l}{x + \sqrt{x^2 - a_e^2 + b_e^2}} - \frac{x}{x + \sqrt{x^2 - a_e^2 + b_e^2}} \\ S &= \frac{b_e^2 + a_e^2}{b_e^2 - a_e^2} - \frac{4b_e^2\, x}{\left[x + \sqrt{x^2 - a_e^2 + b_e^2}\right](b_e^2 - a_e^2)} \end{aligned}\right\} \quad \ldots\ldots(8)$$

In the foregoing expressions for R and S, b_e and a_e are the semi-major and semi-minor axes of the ellipses, respectively; the two elliptic tire prints being identical with respect to size and shape; l is the radius of relative stiffness of the slab [see Table 403(d)].

$$l = \sqrt[4]{\frac{E_c h^3}{12\,(1 - \mu^2)\, k}} \quad \ldots\ldots(9)$$

For concrete pavement, Poisson's ratio, μ, is usually taken as 0.15.

The Westergaard expressions for place and area coefficients and for stresses in the slab utilize the first few terms of infinite series and are therefore applicable for only "points within or near the loaded area" (Westergaard). In the present case, dual tire loading, the expressions are applicable without appreciable error provided that the distance between the ellipses does not exceed the length of the major axis, which is $2b_e$ [Fig. 403(c)]. If the two tires are farther apart than this, it is much more accurate to use the influence chart and the graphical method of Pickett and Ray* which is based on the exact and not the approximate procedure.

The computed stresses at Points *1*, *2*, *3*, etc., of the lower diagram of Fig. 403(c) are plotted in the upper diagram of this figure against the stresses at these several points. Then a smooth curve, as shown in this figure, is drawn through these plotted points. It is preferable to plot the distances of the Points *1*, *2*, *3*, etc., from the origin, in terms of a_e, the semi-minor axis of each tire print. The stress versus distance from the origin curve of the upper diagram has a maximum ordinate. In the diagram shown, this maximum is at a value of x that is slightly less than a_e. The position of the maximum stress necessarily changes with changes in the distance between the loaded elliptical areas, that is, the tire spacing.

(d) *Design application for highway pavement*

The formulas developed by Pickett† are used and two conditions are considered:

1. *Protected corners*—Provision has been made to transfer at least 20 percent of the load from one slab corner to the other by some adequate mechanical means or by aggregate interlock. For this case

*Pickett, Gerald, and Ray, G. K., "Influence Charts for Concrete Pavements," *Proceedings*, ASCE, V. 76, Apr. 1950, Separate No. 12. Charts are available from Kansas State College.

†"Concrete Pavement Design," Portland Cement Association, Chicago, 1951.

$$f_t = \frac{3.36P}{h^2}\left[1 - \frac{\sqrt{a/l}}{0.925 + 0.22\,a/l}\right] \quad \text{.....(10)}$$

See Appendix A401, Fig. A401(a), for design curves based on Eq. (10).

2. *Unprotected corners*—There is no adequate provision for load transfer and one corner must carry over 80 percent of the load.

$$f_t = \frac{4.2P}{h^2}\left[1 - \frac{\sqrt{a/l}}{0.925 + 0.22\,a/l}\right] \quad \text{.....(11)}$$

See Appendix A401, Fig. A401(b), for design curves based on Eq. (11).

In the two foregoing formulas

f_t = maximum tensile stress (psi) at top of slab and in the direction of the bisector of the corner angle

P = wheel load, lb (gross load plus impact). An impact allowance of 20 percent is recommended for pneumatic tires

h = thickness (in.) of a uniform-thickness concrete slab at a corner (or equivalent thickness of a thickened-edge slab)

CHAPTER 5—JOINTS, LOAD TRANSFER DEVICES, AND DETAILS

500—Notation

W = joint width opening, in.

d = diameter of dowel, in.

β = relative stiffness of the dowel and the concrete mass

L_2 = length of the primary bearing pressure wave on dowel, in.

L_T = length of tie-bar, in.

P_D = concentrated load acting downward on dowel at the center of a joint, lb

Q = perimeter of tie-bar, in.

S_b = bearing pressure on concrete under dowel at slab face, psi

A_s' = cross section area of one tie-bar, sq in.

e = base of Napierian logarithms

x = distance along the dowel from slab face at the joint, in.

M_o = bending moment in dowel at slab face, in.-lb; $M_o = (P_D W)/2$

x_m = distance from slab face to point of maximum moment in dowel, in.

501—Definitions

(a) *Dowel*—A plain bar embedded in a concrete pavement to provide vertical shear resistance across a joint.

(b) *Tie bar*—A deformed bar embedded in a concrete pavement at a joint and designed to hold abutting slabs together.

(c) *Joints*

1. Expansion joints

As the term implies, the primary function of an expansion joint is to provide space for the expansion of the pavement and thereby prevent the development of compressive stresses of damaging magnitude. When installed between the pavement and a fixed structure, such as a bridge, an expansion joint serves also to prevent the pavement from exerting damaging pressures against the structure. Expansion joints also function as contraction joints.

2. Contraction joints

The purpose of a contraction joint is to limit tensile stresses to a safe value.

Since contraction joints must be free to open, the continuity of the reinforcement, if reinforcement is present, is interrupted at the joints.

Basically, there are three types of contraction joints, namely, "groove joint," "plate joint," and "butt joint."

a. Groove joint—This joint is created by forming a groove in the upper portion of the pavement. Methods employed to form the groove consist of (1) temporary embedment of a suitable metal strip; (2) installation of a thin strip of premolded joint filler material to the required depth; and (3) sawing the pavement after the concrete has hardened.

In addition to the surface groove, a parting strip of wood, metal, or premolded material is sometimes provided at the bottom of the pavement, directly under the groove, to insure the formation of a vertical crack and to further encourage early cracking of the concrete at the joint.

b. Plate joint—This joint is created by erecting a separator or parting strip on the subgrade. The separator, which usually consists of a metal plate or a thin sheet of some other rigid, noncompressible material, serves to interrupt the continuity of the pavement. A groove is usually formed in the concrete immediately above the separator, to serve as a reservoir for sealing material.

c. Butt joint—This joint occurs where fresh concrete is placed against hardened concrete.

3. Hinge or warping joints

The term "hinge" or "warping joint" applies to any joint which permits hinge action, but no appreciable separation of the adjacent slabs. Hinge joints are intended primarily to relieve warping stresses. In contrast to contraction or expansion joints, appreciable changes in joint width are prevented by the installation of steel tie bars across the joint. In effect, a joint of this type acts simply as a hinge, that is, it merely permits the abutting slabs to undergo a certain amount of vertical angular displacement.

Butt joints become hinge joints when adjacent slabs are held together with tie bars.

4. Construction joints

The term applies to the joint necessitated by any prolonged stoppage of the concreting operations, such as the end of the day's work, equipment breakdown, or delay in delivery of materials.

5. Longitudinal joints

Because of the longitudinal cracking that has occurred extensively in highway pavements constructed with full-width slabs, it has become general practice to divide the pavement into lanes by longitudinal joints.

This joint may be an ordinary butt joint resulting from "lane-at-a-time" construction, or one created during full-width construction by embedding a metal or fiber plate in the concrete along the proposed line of the joint and so shaped as to create mechanical interlocking of the slab edges; or it may be created by weakening the pavement with a deeply formed groove, sawing,

or with an embedded strip of premolded material. Adjacent lanes are usually kept from separating and faulting by properly spaced steel tie bars.

(d) *Mechanical load-transfer devices*

Some form of load transfer should be provided even though the edges and corners of the slabs are designed to carry the load without overstressing the concrete. Loads passing over a joint without load transfer may produce excessive deflections, resulting in permanent damage to the subgrade. This in turn may lead to excessive vertical displacement of slab ends and eventual faulting at joints, even though concrete stresses are not serious.

1. Mechanical load-transfer devices may be divided into two major classifications:

a. Those which develop resistance to shear, but little or no resistance to bending. Included are face-interlock types such as deformed plates and some proprietary makes of load-transfer devices.

b. Those which develop resistance to both shear and bending, of which the dowel bar is the most common. Most proprietary load-transfer devices employ this principle in their design.

2. Mechanical load-transfer devices should possess the following attributes:

a. They should be simple in design in order that they may be practical to install, and permit positive encasement by the concrete.

b. They should be capable of distributing the load stresses throughout the adjacent concrete in such a manner that these stresses will not exceed the allowable design value. In this respect, it is especially important that high localized stresses in the concrete at the joint face be prevented.

c. They should offer no material restraint to the opening of the joints at any time.

d. They should retain their mechanical stability under wheel-load weights and frequencies comparable to those for which the pavement itself has been designed.

e. They should be constructed in such a manner as to meet specified performance requirements relative to load-transfer capacity.

f. They should be resistant to corrosion.

3. The conventional round steel dowel is used to a greater extent than any other type of mechanical load-transfer device. For any given load and pavement condition, it is recommended that all other types of mechanical load-transfer devices be equivalent in load-transfer capacity to that of a round dowel of the dimensions determined in accordance with Sections 502(e) and 503(f).

(e) *Aggregate interlock*—The use of aggregate interlock to transfer loads across joints and to maintain mutual alignment of abutting slabs is generally associated with the construction of nonreinforced pavement and with the use of groove or sawed contraction joints. In order that aggregate interlock may be reasonably effective, the maximum opening of joints should not exceed 0.02 in. except in cases where the pavement is supported by an unusually strong foundation. But even with these precautions, experience

indicates that aggregate interlock frequently is not satisfactory under a large volume of heavy truck traffic.

502—Highway pavements

(a) *Expansion joints*

The width of expansion space per joint and the total expansion space needed are governed by pavement design details, aggregates used, and local climatic conditions. For practical reasons, the maximum width is limited to 1 in. The usual width of opening is ¾ in. To provide more than 1 in. of expansion space at special locations, such as at bridges and railroad crossings, where it is desirable to limit compressive stresses to a safe value, it is recommended that two or three expansion joints of standard width be installed consecutively at intervals of approximately 20 ft. This need apply only in cases where expansion joints are omitted generally elsewhere in the pavement.

Mechanical load-transfer devices are necessary at expansion joints.

At expansion joints, the sliding ends of the dowels must be enclosed in a close-fitting metal cap or thimble to provide space for the movement of the dowel into the concrete during an expansion cycle. The space should be equivalent to the width of the expansion joint, plus ¼ in.

Recommended design details for expansion joints are shown in Fig. 502(a).

(b) *Contraction joints*

Recommended details for contraction joints are set forth in the following paragraphs and illustrated in Fig. 502(b).

1. Groove joint

This type of contraction joint is more widely used than any other. The depth of the surface groove should be not less than 1/6 nor more than ¼ the thickness of the pavement. The depth of the groove may be decreased to 1½ in. if a groove is also formed at the bottom of the pavement by installing a parting strip of suitable height on the subgrade, and in the same vertical plane as the surface groove. The depth of the groove plus the height of the parting strip should not exceed 1/3 the pavement thickness. The width of the groove should be at least twice the anticipated annual variation in joint

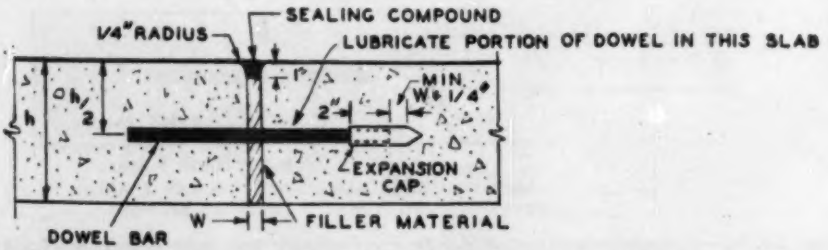


Fig. 502(a)—Recommended design details for highway transverse expansion joint

width, but not less than 1/8 in. This is necessary to facilitate sealing of the joint and to insure that the joint will remain in a sealed condition.

Mechanical load-transfer devices are recommended under all conditions where the joint spacing exceeds 20 ft, and even at a lesser spacing where service conditions are severe.

2. Flat plate joint.

A thin transverse strip of noncompressible material is left in place to form this joint. The strip should be capped in such a manner that when the cap is removed it will form a groove adequate for retention of joint-sealing material. Load-transfer devices are recommended under all conditions of joint spacing.

3. Deformed plate joint

Recently, plates with horizontal deformations or corrugations have been used for the dual purpose of forming the joint and providing mechanical interlock. In the absence of conclusive performance data, no design recommendations can be made at this time. However, the design features of this type of joint are such that it should not be used where slab lengths exceed 20 ft.

4. Butt joint

This is essentially a construction joint, and should be treated in the manner of other types of contraction joints requiring load-transfer devices.

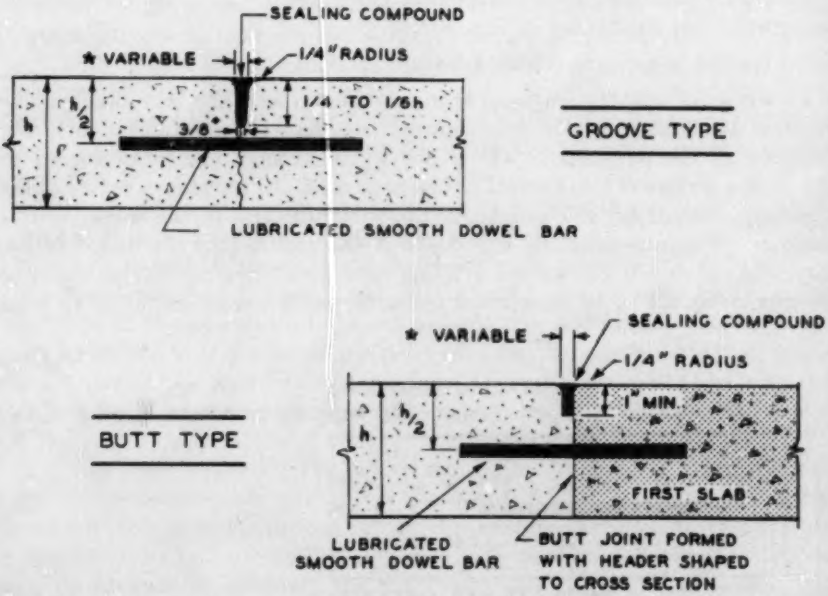


Fig. 502(b)—Recommended design details for highway transverse contraction joints

*Width at least twice annual variation in joint width but not less than ⅛ in.

(c) *Hinge or warping joints*

There are two types of hinge joints, namely, groove and formed tongue-and-groove. They are used mainly in longitudinal joint construction. At hinge joints, separation of the abutting slabs is prevented by bonded steel tie bars or two-piece connectors. Load transfer is accomplished by aggregate or mechanical interlock, and to some extent by the tie bars.

1. Groove joint

Although the formed groove method is more commonly used, the formation of the groove by cutting the pavement with a concrete saw is rapidly taking its place, and is preferentially recommended.

2. Tongue-and-groove joint

The essential elements of this joint are the formed tongue-and-groove, which provide load transfer, and deformed tie bars to hold slabs in adjacent lanes in intimate contact. The tongue-and-groove may be formed in full-

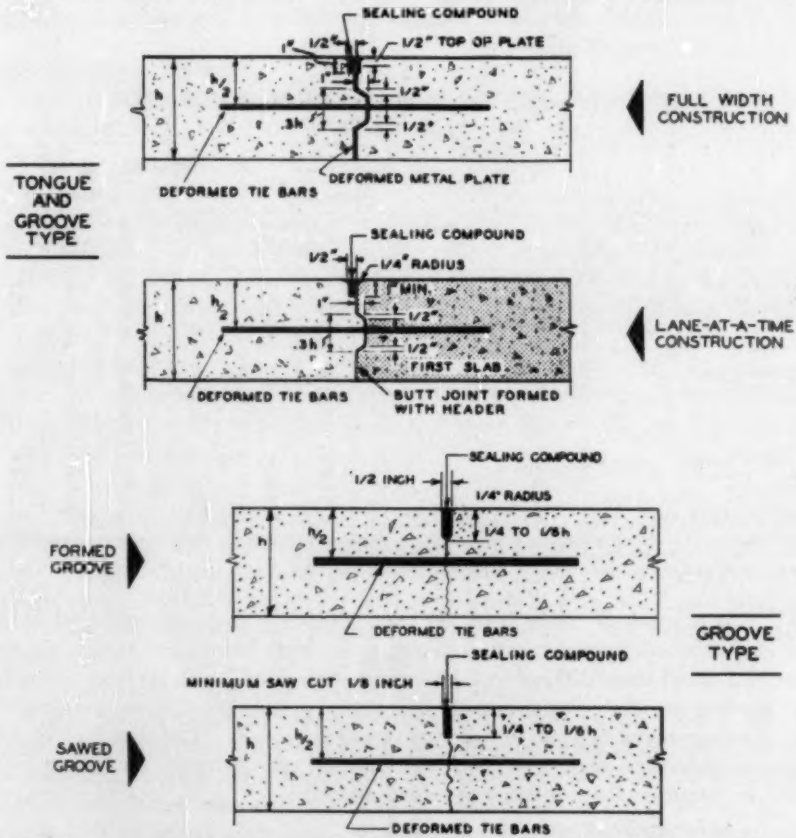


Fig. 502(c)—Recommended design details for highway longitudinal hinge or warping joints

width construction by a preformed metal plate left in place. In lane-at-a-time construction it is recommended that the desired groove in the slab edge be formed by using deformed metal strips or wood strips attached to the forms.

The top edges of the butt type of hinged joint should be finished with a groove so that effective sealing of the joint may be accomplished.

3. Recommended design details for hinge joints are illustrated in Fig. 502(c.).

(d) *Tie bars*

1. The purpose of tie bars is to hold the faces of abutting slabs in intimate contact, thus insuring adequate load transfer. Tie bars are not designed to act as load-transfer devices.

Tie bars are designed to withstand the maximum tensile forces required to overcome the friction between the pavement and the subgrade, from the joint in question to the nearest free edge.

2. Recommended sizes and lengths for different pavement conditions are given in Table 502(d).

TABLE 502(d)—RECOMMENDED MAXIMUM SPACING FOR ½- AND ⅝-IN. TIE BARS*

Type and grade of steel	Working stress, psi	Pavement thickness, in.	½-in. diameter bars				⅝-in. diameter bars			
			Over-all length, in.	Spacing, in.†			Over-all length, in.	Spacing, in.†		
				Lane width 10 ft	Lane width 11 ft	Lane width 12 ft		Lane width 10 ft	Lane width 11 ft	Lane width 12 ft
Structural grade billet or axle steel	22,000	6	20	46	42	38	24	48	48	48
		7		39	36	33		48	48	48
		8		34	31	28		48	48	45
		9		30	28	25		48	43	40
		10		27	25	23		43	39	36
Intermediate grade billet or axle steel	27,000	6	24	48	48	47	27	48	48	48
		7		48	44	40		48	48	48
		8		42	38	35		48	48	48
		9		37	34	31		48	48	48
		10		34	31	28		48	48	44
Rail steel or hard grade billet or axle steel	33,000	6	27	48	48	48	33	48	48	48
		7		48	48	48		48	48	48
		8		48	47	43		48	48	48
		9		46	42	38		48	48	48
		10		41	37	34		48	48	48

*Deformations of tie bars conforming to ASTM A305. Working stress based on 0.67 of minimum yield point strength of steel. Over-all length includes 3-in. allowance for centering.
†It is recommended that spacing of tie bars should not exceed 48 in.

3. The practice of using bent tie bars which are later straightened as in lane-at-a-time construction often results in kinked, broken, or misplaced tie bars. To avoid these difficulties it is recommended that such tie bars be made from electric furnace or open hearth billet steel or axle steel, structural grade, or that an approved type of two-piece connector be used. The tensile strength of such connectors should at least be that of conventional tie bars which would be required for a given pavement design.

(e) *Load-transfer devices*

1. Recommended minimum practical requirements for dowels for various pavement thicknesses are given in Table 502(e).

TABLE 502(e)—RECOMMENDED MINIMUM DOWEL REQUIREMENTS FOR EXPANSION OR CONTRACTION JOINTS IN HIGHWAY CONSTRUCTION

Pavement thickness, in.	Dowel diameter, in.	Dowel length, in.	Dowel spacing, in.
6	¾	18	12
7	1	18	12
8	1	18	12
9	1¼	18	12
10	1¼	18	12

2. Recommendations for load transfer at joints have been based on the most commonly used type of load-transfer device, the ordinary round steel dowel. If proprietary load-transfer devices are used in lieu of dowels, they should have a load-transfer capacity equal to or greater than that of the recommended dowel.

503—Airport pavement

(a) Joint designs applicable to both highway and runway construction covered in Section 502 are omitted here to avoid unnecessary duplication.

(b) *Expansion joints*

1. Longitudinal

Recommended longitudinal expansion joint design details are shown in Fig. 503(b)1. Dowels should not be used at longitudinal expansion joints, but the edges of the slabs along these joints should be thickened to at least 1.25 times the normal slab thickness. Thickness of the joint filler should be ¾ in.

2. Transverse

It is recommended that the thickness of joint filler for transverse expansion joints should be not less than ¾ in. nor more than 1 in. [see Fig. 503(b)2].

(c) *Contraction joints*

1. Groove joint

The groove contraction joint is recommended. The groove shall be of sufficient width and depth to perform its function and to permit proper sealing. In general, the depth of groove may range from 1/6 to ¼ the thickness of the slab. Within these limits the depth shall not be less than 1½ in., nor less than 1/6 the thickness of the slab, nor less than the maximum size of aggregate.

2. Keyed and butt joint (construction joints)

These joints naturally result from lane-at-a-time construction, and may be constructed either with a key (tongue-and-groove interlock), with or without tie bars, or as a plain butt joint with smooth dowel bars. Recommended design details are shown in Fig. 503(b)1.

(d) *Hinge or warping joints*—Depending upon design requirements, hinge or warping joints may be constructed as groove joints, tongue-and-groove joints, or as plain butt joints. In all three cases, tie bars must be employed

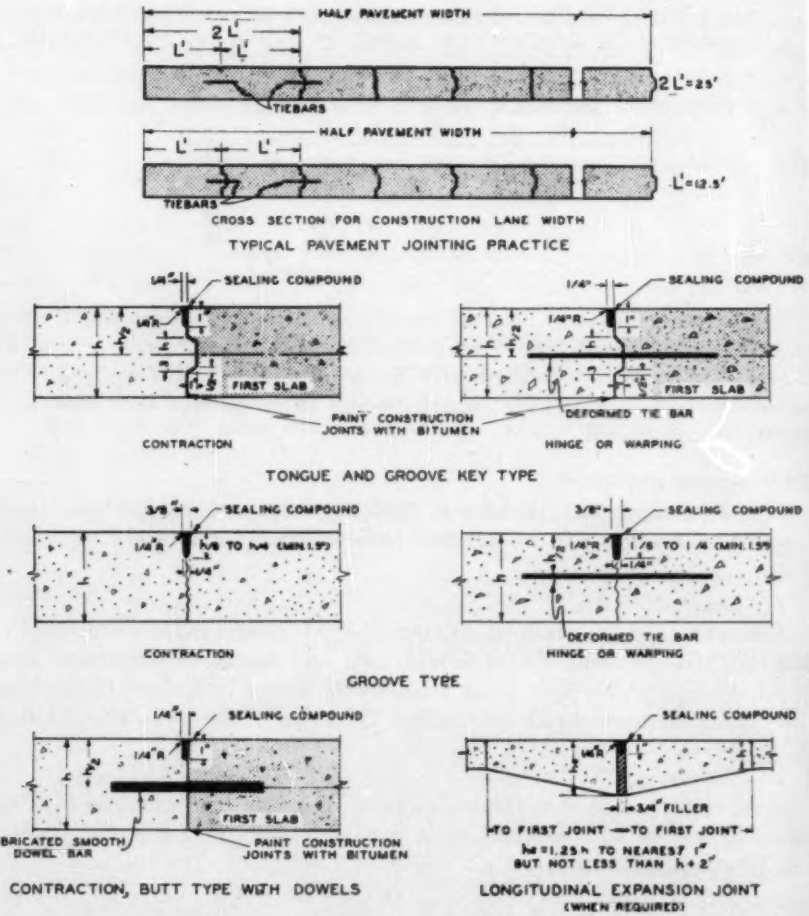


Fig. 503(b)1—Recommended design details for airport runway longitudinal joints

1—Deformed tie bars to be installed in groove and keyed joints occurring within span, 25 ft of free edge of pavement

2—Butt construction joints, with dowels spaced as required, may be used in lieu of keyed joints

3—All joints shall be sealed with approved material

to hold the abutting slabs in intimate contact. Details for this type of joint construction are shown in Fig. 503(b)1.

(e) *Design of tie bars*

1. The recommended minimum diameter of tie bars for airport work is $\frac{5}{8}$ in. The recommended range of tie bar length is 30 to 36 in. at a spacing of 30 in. on center.

2. The selected size and spacing of tie bars shall be such that the cross-sectional area of steel in square inches, per ft of joint, is at least equal to the amount given by the following formula:

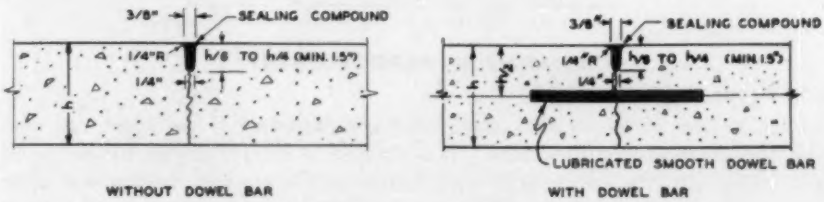
$$A_s = \frac{L' F\, w}{f_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which L' is the distance between the joint in question and the nearest free joint or edge.

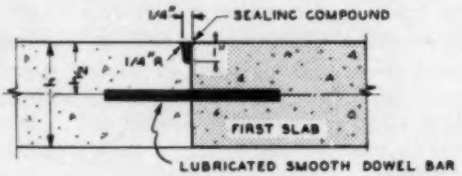
3. A tie bar should be long enough to provide sufficient anchorage on each side of the joint to develop the strength of the tie bar. In addition, an allowance should be made for inaccurate centering of the tie bar. A 3-in. allowance is recommended. Expressed as a formula, this length becomes:

$$L_T = \frac{2 f_s\, A_s}{u\, Q} + 3 \text{ in.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

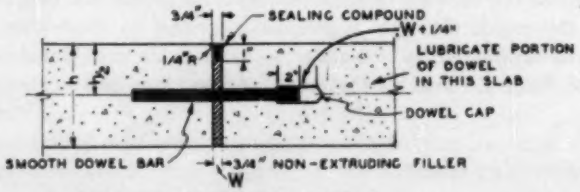
(f) *Load-transfer devices*—Some form of mechanical load transfer is recommended at all transverse expansion and construction joints as well as at butt longitudinal construction joints. The recommended size, length, and spacing of dowels for load transfer are shown in Table 503(f).



GROOVE TYPE CONTRACTION JOINTS



CONTRACTION, BUTT TYPE WITH DOWELS



EXPANSION TYPE WITH DOWELS

Fig. 503(b)2—Recommended design details for airport runway transverse joints

1—Dowels shall be used in groove joints in aprons, taxiways, and thickened end sections of runways. They may be omitted in other portions of runways
2—Dowel size and spacing shall be as required
3—All joints shall be sealed with approved material

TABLE 503(f)—DOWEL REQUIREMENTS FOR LONGITUDINAL BUTT JOINTS, TRANSVERSE EXPANSION, CONSTRUCTION, AND CONTRACTION JOINTS IN AIRPORT CONSTRUCTION

Pavement thickness, in.	Dowel diameter, in.	Dowel length, in.	Dowel spacing, in.
6	¾	18	12
7	¾	18	12
8	1	18	12
9	1	18	12
10	1	18	12
11	1¼	20	15
12	1¼	20	15
13	1¼	20	15
14	1¼	20	15
15	1¼	20	15
16	1½	24	15
17	1½	24	15
18	1½	24	15
19	1½	24	15
20	1½	24	15

CHAPTER 6—REINFORCEMENT

601—Scope

This section presents data and recommendations for the most common type of pavement reinforcement. It is limited to reinforcement in pavement slabs relatively free to undergo contraction and expansion throughout their length. Such slabs are not usually over 100 ft long, although data from continuously reinforced pavements indicate that principles of reinforcement for crack control apply to somewhat greater slab lengths, between 100 and 200 ft. Specially oriented reinforcement, such as edge bars, corner bars, and two-way structural reinforcement used in bridge approach slabs, are not within the scope of this report. The effect of the steel in preventing cracks is not considered herein.

602—General principles

(a) *Purposes of reinforcement*—It is intended to insure the following:

1. Intimate contact of adjacent slabs to produce a degree of load transfer across the crack through aggregate interlock so that wheel loads near the crack are supported by the slabs on both sides with correspondingly smaller concrete flexural stress at the crack and so that load deflections will be minimized.

2. Minimization of water seepage to the subgrade soil and the infiltration of soil into the crack.

(b) *Design criteria*

1. When its temperature drops, a pavement slab tends to shorten. This contraction is resisted by the subgrade. The frictional resistance, which increases from the ends to the center of the slab, must be overcome by the tensile resistance of the steel crossing any cracks. The maximum steel stress

will occur at a crack at mid-length of the slab, and reinforcement is designed on this basis.

2. Frictional resistance will vary with amount of movement, size of slab, rate of temperature change, and characteristics of the subgrade, reaching a maximum when actual sliding of the slab on the subgrade occurs. For design purposes, a coefficient of friction of 1.5 is recommended. From experience, this value has given satisfactory results, probably compensating for other factors not considered in present design practice.

3. A more refined method for determining the coefficient of friction value when sliding and nonsliding conditions are involved has been developed. This method assumes that the subgrade frictional resistance increases in direct proportion to the square root of the slab movement until free sliding takes place, with a constant coefficient of friction for parts of the slab subject to free sliding. A full explanation of this rational method for selection of coefficient of friction values will be found in "Design Considerations for Concrete Pavement Reinforcement for Crack Control," by Subcommittee IV, ACI Committee 325, ACI JOURNAL, Oct. 1956, *Proc.* V. 53, p. 341.

4. The cross-sectional area of steel required per foot of slab width is

$$A_s = \frac{F\,L\,w}{2f_s} \qquad (14)$$

603—Design requirements

(a) *Minimum size of longitudinal reinforcement*—The size of members is dictated by practical considerations and industry standards. Longitudinal reinforcing members in heavy-duty highway pavement should not be less than No. 3 AS&W gage or approximately ¼-in. diameter. In the case of deformed bar mats, the minimum size member should be that of a #3 bar, or ⅜ in. in diameter.

(b) *Transverse reinforcement*

1. Transverse steel reinforcement should be designed to develop a tensile value comparable to the full strength of the tie bars at longitudinal joints in two-lane or multiple lane-pavements.

2. The minimum size of member should be No. 4 AS&W gage or 0.225 in. in diameter. For bar mats, the minimum size of member should be ¼ in. in diameter.

3. From the standpoint of satisfactory cross-welding, the difference in size between a longitudinal and transverse wire should be not greater than six gage numbers.

(c) *Maximum spacing of reinforcement*

1. For welded wire fabric, the spacing of the longitudinal wires should not exceed 6 in., and of the transverse wires, 12 in.

2. For bar mats the spacing of the longitudinal bars should not exceed 15 in., and of the transverse bars, 30 in.

(d) *Location of reinforcement*—Reinforcement should be placed at a depth not less than 2 in. nor more than $\frac{1}{3}\,h$ from top of slab.

APPENDIX—USE OF DESIGN CHARTS

A401—Highway pavement design

(*a*) *Sample design problem*

Required: The pavement thickness for a dual tire load of 11,200 lb (5600 lb on each tire). The center-to-center spacing of the two tires is 13.6 in. and the tire pressure is 75 psi. The value of k is 100. The factor of safety is 2 for a modulus of rupture of 700 of concrete at 28 days. An impact factor equivalent to 20 percent of the load is added.

Solution: The radius, a, of the circular area equal to the area of the equivalent circle for one of the two tires plus the area of a rectangle having a length of 13.6 in. and a width equal to the diameter of the equivalent circle for one of the tires is 10.68 in. In other words, 8.65 in. is the radius of a circle having an area equal to that of a circle of radius 5.34 in., plus the area of a rectangle having a length of 13.6 in. and a width of 10.68 in.

Using the two simplified expressions

$$h = \frac{\sqrt{3.36 \times 13{,}440\ B}}{350}, \text{ protected corners} \quad (15)$$

$$h = \frac{\sqrt{4.20 \times 13{,}440\ B}}{350}, \text{ unprotected corners} \quad (16)$$

where B is the quantity in brackets in Eq. (10) and (11), it is found by trial that h is 8 in. for protected corners and 9.5 for unprotected corners. The trial method is illustrated as follows. For the case of unprotected corners, for example, a value is arbitrarily assigned to h. Then B is computed and the expression under the square root sign in the expression on the right is computed. The procedure is repeated until the value for h and this expression are made numerically equal.

(*b*) *Use of design charts*

1. Assuming load conditions as stated above, the design axle load would be 13,440 × 2 or 26,880 lb. For the case of protected corners, enter design chart Fig. A401(a) for axle load on left margin at load of 26,880 lb and proceed right horizontally to the design curve line for $k = 100$. Proceed vertically downward to bottom edge of chart and read the required thickness, which in this example, is approximately 8 in. For unprotected corners, design chart Fig. A401(b), the thickness would be approximately 9¼ in.

A402—Airport pavement design*

(a) *Use of the airport design charts for determining pavement thickness*

1. The proper working stress should be established. This is done by dividing the estimated modulus of rupture of the concrete by the appropriate safety factor.

2. The k value of the subgrade must be determined as explained previously.

3. The type of landing gear, gear load, and tire pressure must be known for the planes which are anticipated on the facility being designed.

*Design charts from "Design of Concrete Airport Pavement," Portland Cement Association, Chicago, 1955.

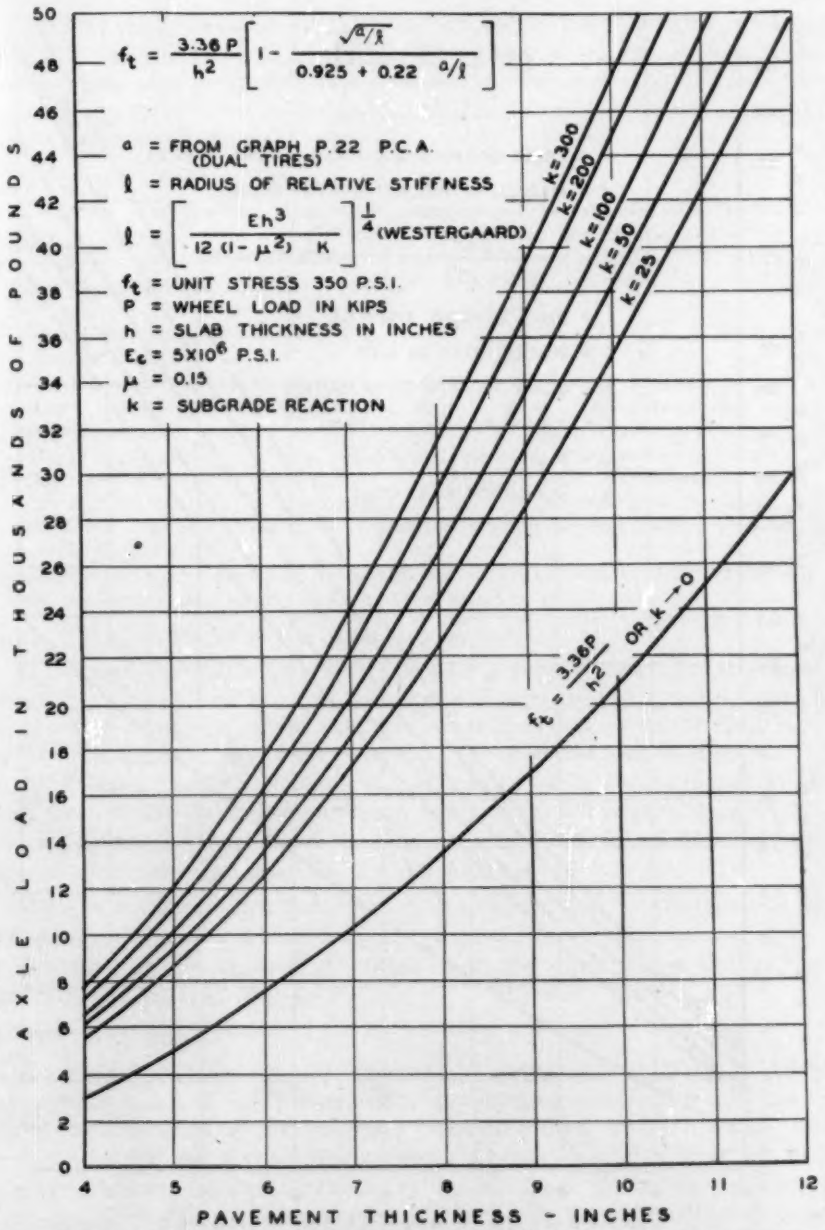


Fig. A401(a)—Design chart based on Eq. (10) for portland cement concrete pavements —protected corners*

*For modulus of rupture values greater or less than 700 psi, see design chart (Fig. 4) in "Concrete Pavement Design," Portland Cement Association, 1951.

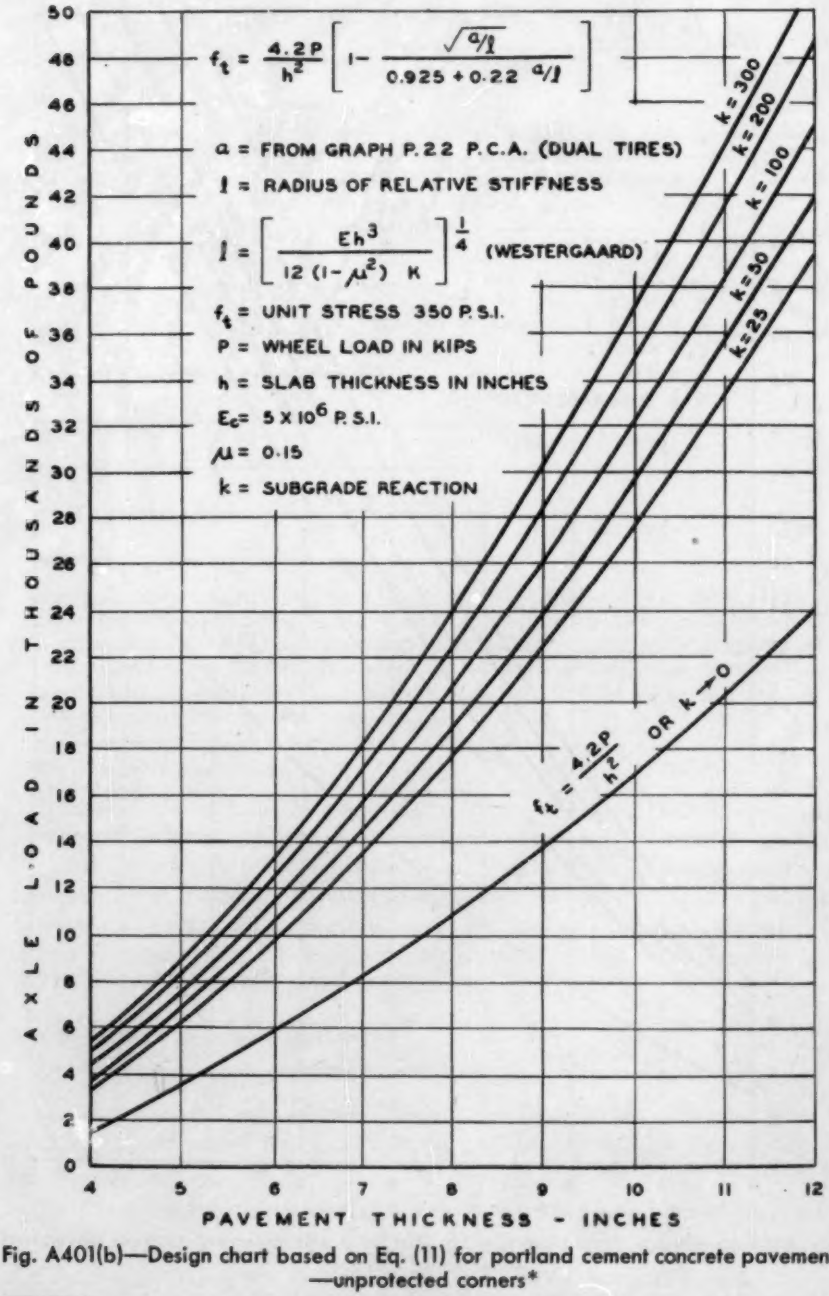


Fig. A401(b)—Design chart based on Eq. (11) for portland cement concrete pavements —unprotected corners*

*For modulus of rupture values greater or less than 700 psi, see design chart (Fig. 5), *op. cit.*

4. Enter the appropriate chart at the left margin with allowable working stress and proceed from left to right in the manner of the broken lines with arrows on the charts—read right from allowable working stress to curve for k value, then up or down to design-load line, and from that point read right to slab thickness at the right edge.

(b) *Sample design problem*

Required: The thickness of an apron extension and taxiway for an occasional plane having a dual-wheel load of 80,000 lb. The existing pavement is designed for a 40,000-lb dual-wheel load, and aircraft with loads of this magnitude are making several operations per day. The k of subgrade established from tests on the existing subgrade under the old apron is 200 lb per cu in., and the concrete mix is designed for a 28-day modulus of rupture of 650 psi.

Solution:

1. Allowable working stress equals the modulus of rupture, increased to the 6-month strength (650 × 114 percent = 741 psi) divided by safety factor (say 1.7 for the occasional 80,000-lb dual) or 741 divided by 1.7, which equals 436 psi.

2. Enter the design chart, Fig. A402(b) for dual-wheel loads, on the left margin at a stress of 436 psi and proceed right horizontally to the desired curved line for k 200.

3. Proceed vertically to the diagonal line for the dual-wheel load in question (80,000 lb). The heavy solid lines indicate average tire pressures commonly used for the load given.

4. Proceed right horizontally to the edge of the chart and read the required thickness, in this example, 10.8 in. Probably 11 in. would be used.

(c) *Use of charts for review of existing pavement*

(1) The design charts may also be used in reverse to determine the stress caused by a given load in a pavement of a known thickness and on a subgrade whose k is known. For example, use of the charts could be illustrated by checking the adequacy of the 11 in. obtained above for unlimited operations of the 40,000-lb dual. Starting at 11 and moving left across to the 40,000-lb load, down to the 200 k line, and horizontally to the left edge, a stress of 238 psi or a safety factor of 2.8 at 28 days is obtained. This indicates that the pavement is more than adequate for this loading.

Loads or k values other than those shown can be used by interpolation on the charts.

The same procedure would be followed on the other design charts, Fig. A402(a), (c), (d), and (e).

(d) *Tire pressure*

The dashed diagonal lines are included to permit the use of these charts for a wide range of tire pressures for any landing-gear load. The heavy diagonal lines indicate tire pressures commonly used for each load. For example, in Fig. A402(a) for single-wheel loads with a load of 75,000 lb, the average pressure indicated by the heavy line is 100 psi. However, the dashed lines permit the use of tire pressures of 120 psi or 80 psi if either of these values should be desired. Intermediate values may be interpolated.

Fig. A402(d) and A402(e) were prepared for military jet aircraft and include the range of tire pressures common for these planes.

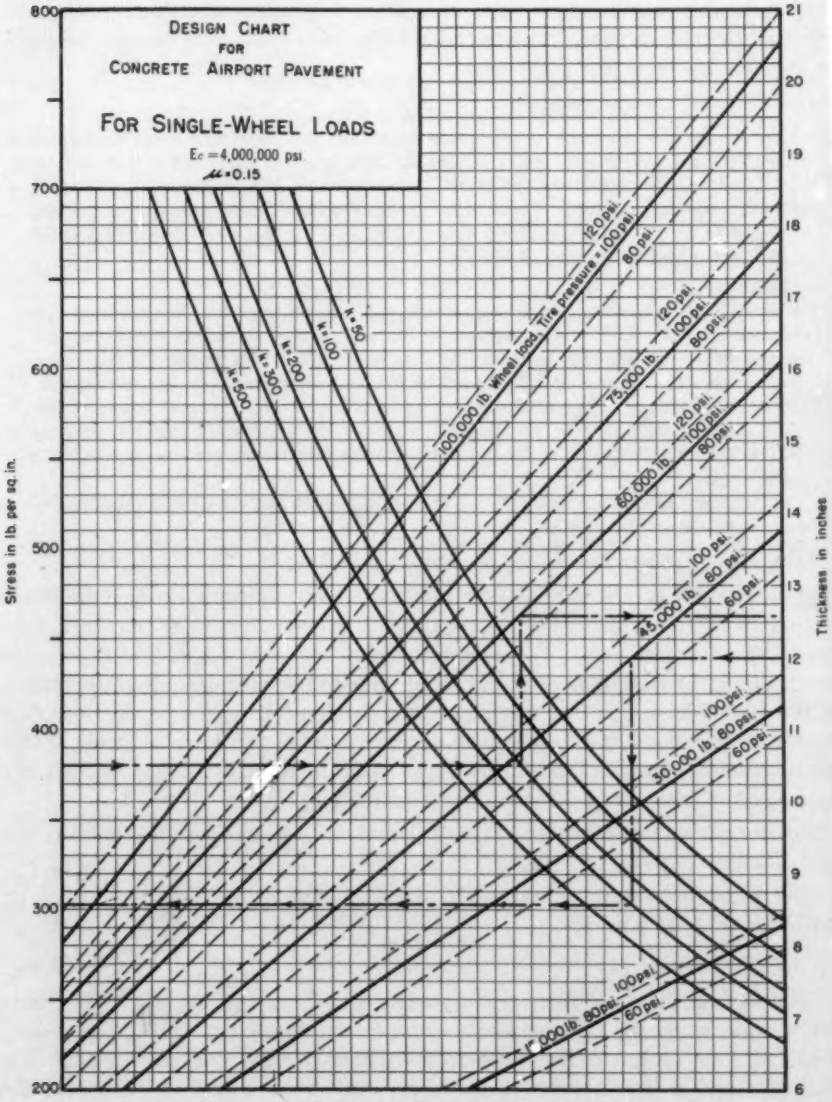


Fig. A402(a)*

*Fig. 4, "Design of Concrete Airport Pavement," Portland Cement Association, 1955.

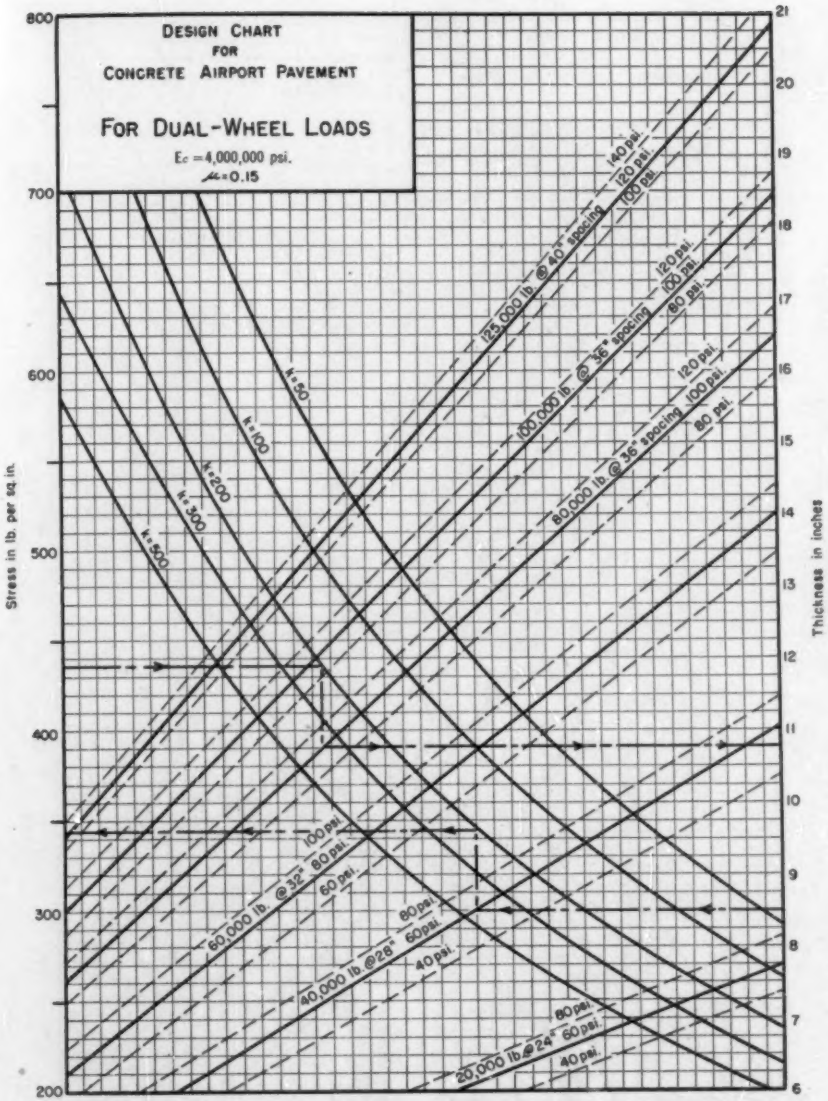


Fig. A402(b)*

*Fig. 5, *op. cit.*

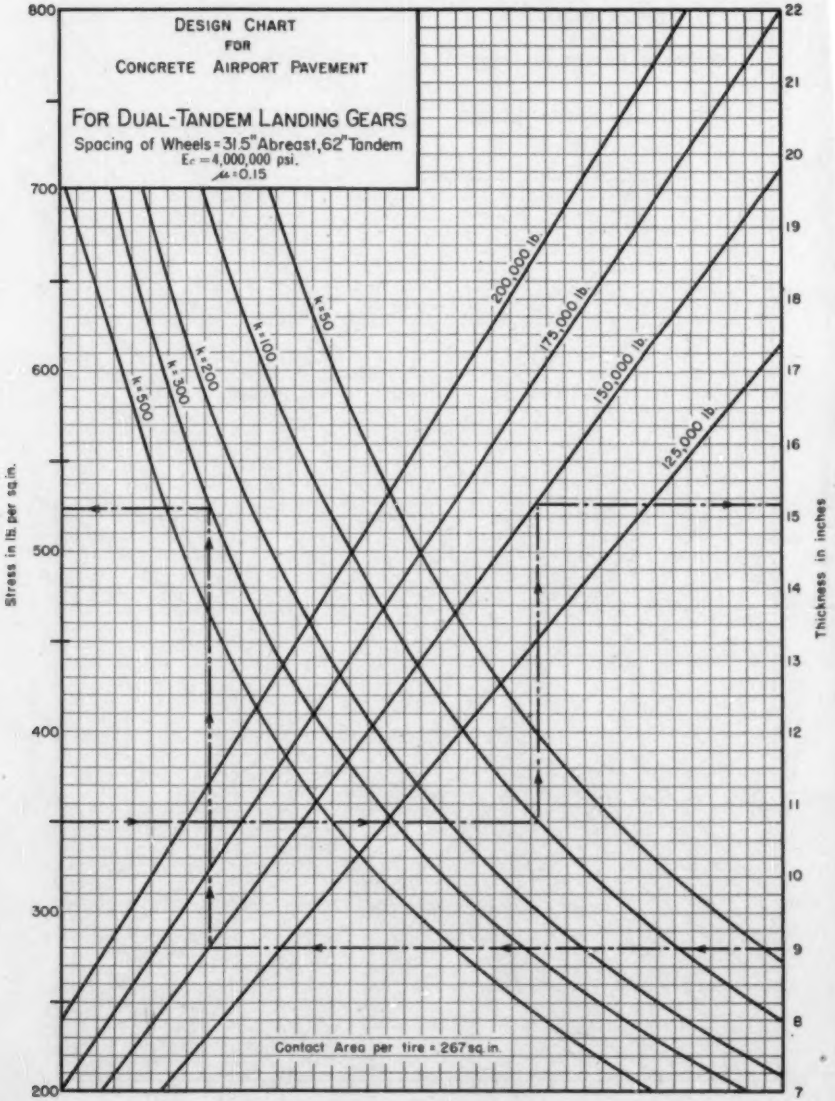


Fig. A402(c)*

*Fig. 6, *op. cit.*

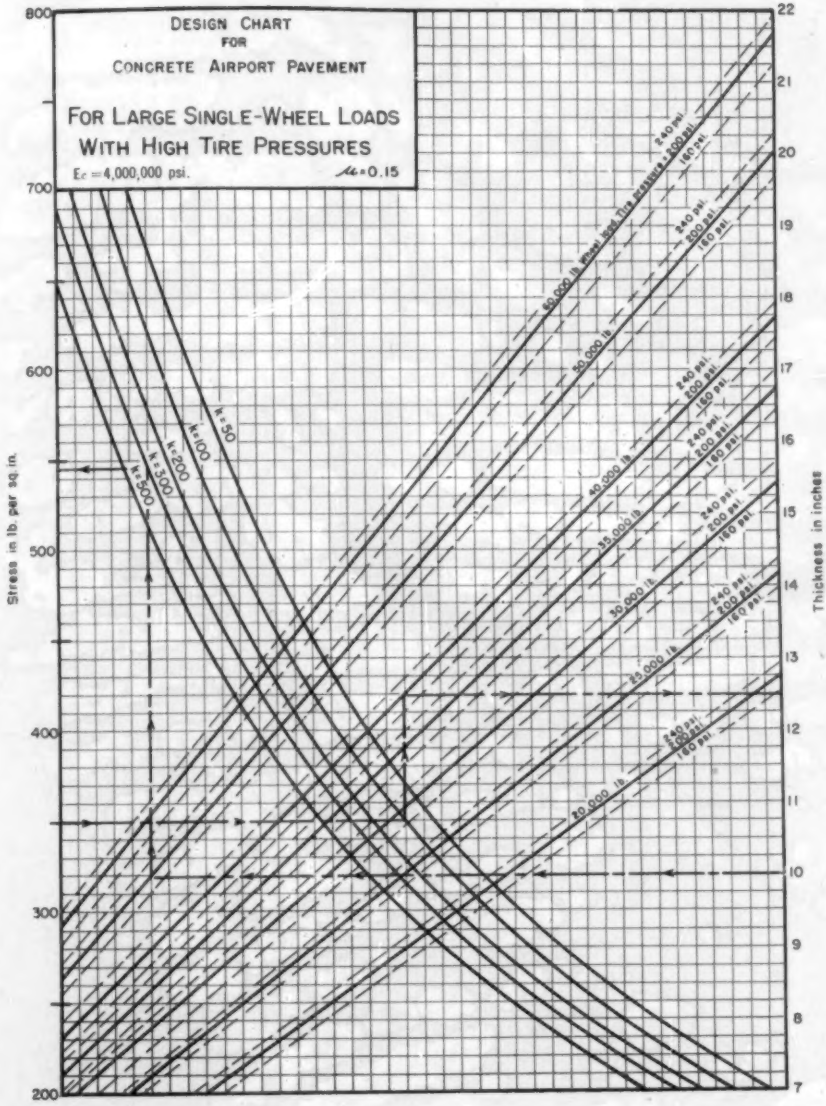


Fig. A402(d)*

*Fig. 7, *op. cit.*

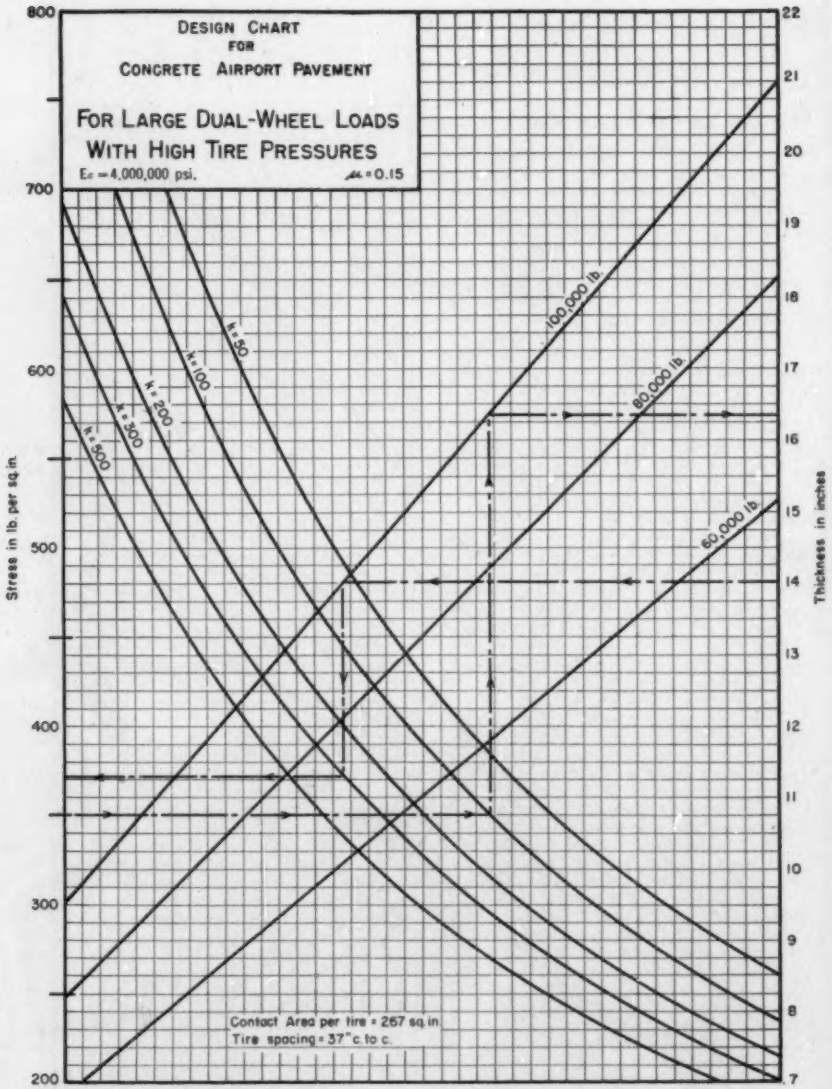


Fig. A402(e)*

*Fig. 8, *op. cit.*

(e) *Other landing gear*

Where the thickness is required for a dual-wheel landing gear with a specific spacing other than those shown, the required thickness may be obtained from Fig. A402(a) and A402(e) by applying the following approximate correction factors:

1. For increases in center-to-center tire spacing up to 10 in., the required thickness should be reduced by 0.6 percent for each inch increase.

2. For each inch decrease in dual spacing up to 10 in., the required thickness should be increased by 0.6 percent.

Similar correction factors cannot be applied for dual-tandem landing gears other than that given in Fig. A402(c). If the spacings are changed appreciably, a special design chart can be developed.

Discussion of this report should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

ACI Standard

Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)*

Reported by ACI Committee 617

H. F. CLEMMER
Chairman

E. A. ABDUN-NUR
HAROLD ALLEN
A. A. ANDERSON
E. W. BAUMAN
JOHN A. BISHOP
FRANK B. BROWN
A. T. GOLDBECK
W. E. HAWKINS
T. E. HOWELL
FRED HUBBARD
T. J. KAUER
PHILLIP L. MELVILLE
BAILEY TREMPER
STANTON WALKER

SYNOPSIS

Specifications apply to construction of portland cement concrete pavements and bases under normal conditions for both highways and airports. Standards and specifications of several other organizations are incorporated by reference.

Sources and acceptance standards for materials are given, as well as materials testing procedures and procedures for test of concrete strength to be used as a basis for proportioning of mixtures. Specifications for the materials and construction of soil foundations for concrete pavements and concrete bases are included. Provision is made for use of foundations stabilized by a cementing agent, but materials and construction of such are beyond scope of this specification.

Materials, dimensions, setting, and removal of forms are treated. Construction methods are specified for forming joints, installation of joint seal and load transfer devices, and placing of reinforcement. Concrete proportioning based on design for minimum strength is covered in detail; proportioning based on fixed cement content is allowed.

Other sections cover production of high-early-strength concrete and the handling and mixing of materials. Detailed requirements are given for placing, finishing, and curing of pavement concrete. Check of thickness of finished pavement is cited as basis for adjustments in payment to contractor. Protection of finished pavement, opening it to traffic, and public use of thoroughfare during construction are also specified.

*Adopted as a standard of the American Concrete Institute at it its 54th annual convention, Feb. 25, 1958, as reported by Committee 617; ratified by letter ballot June 1, 1958. Title No. 55-3 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Title No. 55-3 supersedes Title No. 47-49 published in May 1951. Separate prints in covers are available at 50 cents each to ACI members ($1.00 to nonmembers). **Discussion** of this report (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

CHAPTER 1—INTRODUCTION

101—Scope

(a) These specifications apply to the construction of portland cement concrete pavements and bases under normal conditions for both highways and airports, including the preparation of the subgrade, and shall govern unless modified by special provisions to take into account unusual conditions of traffic, subgrade, drainage, exposure, and other factors.

(b) Subjects covered include: materials; proportions of materials based on design for minimum strength or based on uniform cement factor; measurement and handling of materials; mixing; high-early-strength concrete; subgrade and subbase preparation; forms; installation of joints and reinforcement; placing and finishing concrete; and curing.

102—Reference specifications cited in this recommendation

The specifications of the American Society for Testing Materials, American Association of State Highway Officials, the federal government, and standards of the American Concrete Institute referred to in these specifications are listed below with their serial designations including the year of latest revision where applicable. These specifications or the latest revisions thereof are declared to be a part of this specification, the same as if fully set forth elsewhere herein:

ACI standards

Recommended Practice for the Design of Concrete Pavements (ACI 325-58)
Recommended Practice for Winter Concreting (ACI 604-56)

ASTM specifications

A 15-54T	Tentative Specifications for Billet-Steel Bars for Concrete Reinforcement
A 16-54T	Tentative Specifications for Rail-Steel Bars for Concrete Reinforcement
A 160-54T	Tentative Specifications for Axle-Steel Bars for Concrete Reinforcement
A 184-37	Standard Specifications for Fabricated Steel Bar or Rod Mats for Concrete Reinforcement
A 185-54T	Tentative Specifications for Welded Steel Wire Fabric for Concrete Reinforcement
A 305-53T	Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement
C 10-54	Standard Specifications for Natural Cement
C 31-49	Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field
C 33-55T	Tentative Specifications for Concrete Aggregates
C 39-49	Standard Method of Test for Compressive Strength of Molded Concrete Cylinders
C 78-49	Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
C 94-55T	Tentative Specifications for Ready-Mixed Concrete
C 127-42	Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate
C 128-42	Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate
C 138-44	Standard Method of Test for Weight per Cubic Foot, Yield, and Air Content (Gravimetric) of Concrete

C 143-52	Standard Method of Test for Slump of Portland-Cement Concrete
C 150-55	Standard Specifications for Portland Cement
C 171-53	Standard Specifications for Waterproof Paper for Curing Concrete (Tentative Revision of Specifications C 171 issued June 1955)
C 172-54	Standard Method of Sampling Fresh Concrete
C 173-55T	Tentative Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method
C 174-49	Standard Method of Measuring Length of Drilled Concrete Cores
C 175-55	Standard Specifications for Air-Entraining Portland Cement
C 192-55	Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory
C 205-53T	Tentative Specifications for Portland Blast-Furnace Slag Cement
C 231-54	Standard Method of Test for Air Content of Freshly Mixed Concrete by Pressure Method
C 260-54	Standard Specifications for Air-Entraining Admixtures for Concrete
C 309-53T	Tentative Specifications for Liquid Membrane-Forming Compounds for Curing Concrete
C 340-55T	Tentative Specifications for Portland-Pozzolan Cement
C 350-54T	Tentative Specifications for Fly Ash for Use as an Admixture in Portland Cement
C 358-55T	Tentative Specifications for Slag Cement
C 360-55T	Tentative Method of Test for Ball Penetration in Fresh Portland Cement Concrete
D 98-56T	Standard Specifications for Calcium Chloride
D 544-52T	Tentative Specifications for Preformed Expansion Joint Fillers for Concrete (Nonextruding and Resilient Types)

Federal specifications

SS-S-164	Sealer, Hot-Poured Type, for Joints in Concrete
SS-S-156	Sealer, Cold-Application Emulsion Type, for Joints in Concrete
SS-S-159	Sealer, Cold-Application Mastic Type, for Joints in Concrete

AASHO specifications

M 73-49	Standard Specifications for Cotton Mats for Curing Concrete
M 74-55	Standard Specifications for Subgrade Paper
M 139-54	Standard Specifications for Waterproof Paper for Curing Concrete
M 147-55	Standard Specifications for Materials for Soil Aggregate Subbase, Base, and Surface Courses
T 26-51	Quality of Water to be Used in Concrete
T 89-54	Standard Methods of Determining the Liquid Limit of Soils
T 91-54	Standard Methods of Calculating the Plasticity Index of Soils
T 99-49	Standard Laboratory Method of Test for the Compaction and Density of Soil

CHAPTER 2—MATERIALS

201—Sources

Cement, aggregates, admixtures, and water shall be furnished only from sources of supply approved by the engineer before shipments are started, and may be used only so long as the materials meet the requirements of these specifications. The basis of approval of such sources shall be the ability to produce materials of the quality and in the quantity required.

202—Requirements for materials

(a) *Admixtures*—Materials other than cement, fine aggregate, coarse aggregate, and water for use in concrete pavement shall conform to the following specifications:

1. "Specifications for Air-Entraining Admixtures for Concrete" (ASTM C 260).

2. "Specifications for Fly Ash for Use as an Admixture in Portland Cement Concrete" (ASTM C 350).

3. "Specifications for Calcium Chloride" (ASTM D 98).

(b) *Aggregates*

1. Aggregates shall conform to the requirements of "Specifications for Concrete Aggregates" (ASTM C 33).

Coarse aggregate shall be furnished in at least two separate sizes with the separation at the ¾-in. sieve when combined material graded from No. 4 to 1½ in. is specified and at the 1-in. sieve when combined material graded from No. 4 to 2 in. is specified.

2. Aggregates shall be so stored as to prevent the inclusion of foreign material. Aggregates shall not be stored upon the finished subgrade. Aggregates of different kinds and sizes shall be placed in separate stockpiles. Stockpiles of coarse aggregates shall be built up in successive horizontal layers not more than 3 ft thick. Each layer shall be completed before the next is started. Should segregation occur, the aggregates shall be remixed to conform to the grading requirements.

3. Frozen aggregates or aggregates containing frozen lumps shall be thawed before use. Washed aggregates and aggregates produced or manipulated by hydraulic methods shall be allowed to drain for at least 12 hr before use. Stockpiles or cars and barges equipped with weep holes are considered to offer suitable opportunity for drainage.

(c) *Cement*—Cement for use in concrete pavements shall be as specified and shall conform to the requirements of applicable ASTM specifications listed.

1. Cement may consist of any of the following types or combinations thereof:
   a. Portland cement (ASTM C 150)
   b. Air-entraining portland cement (ASTM C 175)
   c. Portland blast-furnace slag cement (ASTM C 205)

2. Cement may consist of a blend of portland-type cement as above and one of the following other cements:
   a. Natural cement (ASTM C 10)
   b. Slag cement (ASTM C 358)
   c. Portland-pozzolan cement (ASTM C 340)

(d) *Cover materials for curing*

1. Burlap shall be made from jute or hemp and, at the time of using shall be in good condition, free from holes, dirt, clay, or any other substance which interferes with its absorptive quality. It shall not contain any substance which would have a deleterious effect on the concrete. Burlap shall be of such quality that it will absorb water readily when dipped or sprayed and shall

weigh not less than 7 oz per sq yd when clean and dry. Burlap made into mats may be used if care in handling is exercised to avoid marring the finished surface of the concrete.

2. Cotton mats for curing concrete shall conform to the requirements of "Specifications for Cotton Mats for Curing Concrete" (AASHO M 73).

3. Paper for curing concrete shall conform to the requirements of "Specifications for Waterproof Paper for Curing Concrete" (ASTM C 171), (AASHO M 139).

4. Impermeable sheets other than paper shall conform to the water retention requirements of "Specifications for Waterproof Paper for Curing Concrete" (ASTM C 171).

5. Liquid membrane-forming compounds shall conform with the requirements of "Specifications for Liquid Membrane-Forming Compounds for Curing Concrete" (ASTM C 309).

(e) *Expansion joint filler*—Expansion joint filler shall conform to the requirements of "Specifications for Preformed Expansion Joint Fillers for Concrete" (ASTM D 544).

(f) *Joint sealing material*—Joint sealing material shall conform to the requirements of "Federal Specifications for Sealer; Hot-Poured Type, for Joints in Concrete" (SS-S-164); "Sealer, Cold-Application Emulsion Type, for Joints in Concrete" (SS-S-156); or "Sealer, Cold-Application Mastic Type, for Joints in Concrete" (SS-S-159).

(g) *Metal joint plate and pins*—If plates are used for tongue-and-groove joints, they shall be made of metal not less than 16 gage (manufacturers' standard gage for steel sheets) in thickness. Each section shall be a continuous strip of metal not less than 10 ft long having a width ½ in. less than the depth of the pavement and shall be provided with an end connection which will hold the ends of strips firmly together. Each section shall be of the specified cross section, and shall be punched for dowels or tie bars, and pins, as specified or shown on the plans.

Pins shall be channel shaped, pressed out of sheet steel of not less than 12 gage (manufacturers' standard gage for steel sheets) in thickness; they shall be a minimum of 15 in. in length or longer if necessary to provide proper support.

(h) *Reinforcing steel and accessories*

1. Steel wire fabric reinforcement shall conform to the requirements of "Specifications for Welded Steel Wire Fabric for Concrete Reinforcement" (ASTM A 185).

2. Bar mats—The steel in bar mats shall conform to the requirements of "Specifications for Bar or Rod Mats for Concrete Reinforcement" (ASTM A 184). Members shall be of the size and spacing shown on the plans. All intersections of longitudinal and transverse bars shall be securely wired, clipped, or welded together in the plant of the steel supplier.

3. Bars—Reinforcing bars shall conform to the requirements for intermediate or hard grade in "Specifications for Billet-Steel Bars for Concrete Reinforcement" (ASTM A 15), or of "Specifications for Rail-Steel Bars for Con-

crete Reinforcement" (ASTM A 16), or of "Specifications for Axle-Steel Bars for Concrete Reinforcement" (ASTM A 160). Bars depending upon bond for their effectiveness shall conform to the requirements of "Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement" (ASTM A 305), and shall be free from excessive rust, scale, or other substances which prevent the bonding of the concrete to the reinforcement.

4. Tie bars shall be deformed steel bars conforming to the requirements of the specifications for reinforcing bars except that structural-grade steel bars, only, shall be used where they are to be bent and restraightened. Joint hook bolts may be used as an alternate to tie bars. Such bolts shall not be less than ½ in. in diameter and should be equipped with threaded couplings.

5. Dowels and sleeves shall be plain round bars conforming to the requirements of the specifications for reinforcing bars. Dowel bars shall not be burred, roughened, or deformed out of round in such a manner as to affect slippage in the concrete.

When metal sleeves are used, they shall cover the ends of the dowels for not less than 2 in. nor more than 3 in. The sleeve shall be closed at one end and shall have a suitable stop to hold the end of the bar at least 1 in. from the closed end of the sleeve. It shall be of such rigid design that the closed end will not collapse during construction.

6. Chairs for holding tie rods in correct position while the concrete is being placed shall be made of metal, minimum 16 gage (manufacturers' standard gage for steel sheets) in thickness, a minimum of 11 in. in length and 1⅛ in. in width. This material shall be slightly rounded in the 1⅛-in. dimension and shall be tapered slightly at the bottom to facilitate driving into the finished grade.

7. Stakes used to support expansion joint fillers shall be channel or U-shaped metal, ¾ in. wide, ⅜ in. deep, and not less than 16 gage (manufacturers' standard gage for steel sheets) in thickness. They shall be a minimum of 15 in. in length or longer if necessary to provide proper bearing support.

(i) *Subgrade paper*—Paper to be used under the slab shall conform to the requirements of "Specifications for Subgrade Paper" (AASHO M 74).

(j) *Water*—Water used in mixing or curing concrete shall be clean and free from injurious amounts of oil, salt, acid, vegetable, or other substances harmful to the finished product. Sources of water shall be maintained at such a depth, and the water shall be withdrawn in such a manner (by enclosing pump intake, etc.), as to exclude silt, mud, grass, or other foreign materials. Water shall be secured from previously approved sources or sources approved after testing in accordance with "Quality of Water to be Used in Concrete" (AASHO T 26).

CHAPTER 3—TEST OF MATERIALS

301—Test methods

Materials shall be tested in accordance with methods referred to in the appropriate specifications, except as otherwise specified.

302—Flexural strength tests of concrete as basis of design

Specimens for flexural strength tests to be used as the basis for the design of concrete mixtures shall be molded and cured in accordance with "Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory" (ASTM C 192). Specimens shall be tested in accordance with "Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)" (ASTM C 78).

303—Flexural strength tests of concrete

(a) Two groups of flexural strength specimens shall be made in the field in accordance with "Method of Making and Curing Concrete Compression and Flexure in the Field" (ASTM C 31). One of these groups shall be for curing in accordance with Sections 7(a) and (b) of that standard and one for curing in accordance with Sections 7(a) and (c). The first group shall be for the purpose of checking the adequacy of the laboratory design for the conditions of proportioning and mixing on the job. The second group shall be for checking the adequacy of curing and other job conditions which determine when the structure may be put in service.

(b) Specimens for checking laboratory design shall be tested in accordance with "Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)" (ASTM C 78). Specimens tested after curing in the field should be tested in accordance with ASTM C 78. If another type of testing machine is used, results with it should be correlated with those attained from the standard apparatus.

(c) Number of specimens—At least three beams, for each of the two curing methods, shall be made for at least each 2000 sq yd of pavement or a portion thereof placed on any one day.

304—Compressive strength of concrete

The compressive strength test specimens of concrete for use in design shall be made and cured in accordance with "Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory" (ASTM C 192). They shall be tested in accordance with "Method of Test for Compressive Strength of Molded Concrete Cylinders" (ASTM C 39).

305—Specific gravity and absorption of aggregates

(a) *Fine aggregate*—The bulk specific gravity in a saturated surface dry condition and absorption of fine aggregate shall be determined in accordance with "Method of Test for Specific Gravity and Absorption of Fine Aggregates" (ASTM C 128).

(b) *Coarse aggregate*—The bulk specific gravity in a saturated surface dry condition and absorption of coarse aggregate shall be determined in accordance with "Method of Test for Specific Gravity and Absorption of Coarse Aggregate" (ASTM C 127).

306—Air content

The air content of plastic concrete shall be determined in accordance with ASTM standard methods of test for air content: (1) gravimetric, C 138; (2) volumetric, C 173; or (3) pressure method, C 231. For concretes made with blast furnace slag or similar aggregates, the volumetric (rolling) method shall be used.

307—Consistency

(a) Consistency shall be determined in accordance with "Method of Slump Test for Consistency of Portland Cement Concrete" (ASTM C 143) or "Method of Test for Ball Penetration in Fresh Portland Cement Concrete" (ASTM C 360).

(b) When the penetration method is used for field control, specified slumps shall be converted to equivalent penetrations by factors established in advance during tests as a basis for design.

CHAPTER 4—SOIL AND SUBBASE FOUNDATION PREPARATION

401—Scope

Specifications for the materials and construction of soil foundations for concrete pavements and concrete bases are given. Provision is made for the use of foundations stabilized by a cementing agent, but materials and construction of such foundations are not within the scope of this specification.

402—Definitions

The following definitions shall apply in these specifications where applicable:

Embankment foundation—The material on which an embankment is placed.

Embankment (fill)—A raised structure of soil, soil-aggregate, or rock.

Subgrade—The prepared and compacted soil below the pavement system.

Subbase—The layer used in a pavement system between the subgrade and the base course.*

Soil-aggregate base course†—The layer used in a pavement system to reinforce or protect the subgrade or subbase.

Concrete base course—The layer used in a pavement system to reinforce and protect the subgrade or subbase, and to support a wearing course of other material.

Silt-clay (minus No. 200 material)—Fine soil particles which will pass the No. 200 sieve.

(a) *Silt fraction*—Material passing the No. 200 sieve and larger than 0.005 mm.

(b) *Clay fraction*—Material smaller than 0.005 mm including colloids which are the particles of the clay fraction smaller than 0.001 mm.

403—Soil fill material

All material used in earth fills must meet the following specifications and may be rejected on visual inspection pending the testing of samples.

(a) Fill material shall be a well-graded combination of granular material and silt-clay soil. The material shall have at least 60 percent retained on,

*Concrete pavement is usually placed directly upon the subgrade or subbase and performs both the functions of a wearing surface and base.

†When a special soil-aggregate base course separate from the subbase is specified for use under a concrete pavement, the base course material shall conform to the requirements for subbase material of "Specifications for Materials for Soil-Aggregate Subbase, Base, and Surface Courses" (AASHO M 147).

and at least 10 percent passing the No. 200 sieve. No gravel or stone shall be larger than one-third the depth of the layer to be placed. The soil shall have a liquid limit, as determined by "Methods of Determining the Liquid Limit of Soils" (AASHO T 89), not greater than 40 and a plasticity index as determined by "Method of Calculating the Plasticity Index of Soils" (AASHO T 91) of not more than 10. The material to be used shall be free from trash, brick, broken concrete, tree roots, sod, ashes, or cinders.

(b) Materials other than specified may be used if approved by the engineer and the required densities are attained.

(c) Samples will be taken whenever in the opinion of the engineer the texture of the material appears to be changing.

(d) Contractors desiring approval of the sources of a material shall submit samples at least one week in advance of the desired date of approval. Acceptance of material from any location shall not be construed as approval of the entire location but only insofar as it continues to meet the specifications.

404—Subbases

(a) *Material*—The subbase material shall meet the requirements as specified in "Specifications for Materials for Soil-Aggregate Subbase, Base and Surface Courses" (AASHO M 147).

(b) *Gradation*—The subbase material shall be well graded. Of the material passing the No. 10 sieve, not more than 25 percent should pass the No. 200 sieve. Also the percentage of material passing the No. 200 sieve should not be greater than two-thirds the percentage passing the No. 40 sieve.

(c) *Plasticity requirements*—The portion of the subbase material that passes the No. 40 sieve shall have a maximum liquid limit of 25 and a maximum plasticity index of 6.

(d) *Non-frost susceptibility*—Where freezing of soil moisture under the pavement is probable, the subbase shall be of a non-frost susceptible material. Gradation requirements of non-frost susceptible materials vary from a maximum of 8 percent finer than 0.02 mm for uniform sands to 3 percent finer than 0.02 mm for other nonuniform granular material.

(e) *Stabilized subbases*—Granular or soil materials adequately stabilized with a cementing agent may also be used as directed by the engineer.

405—Compaction

(a) *Equipment*—Compaction may be accomplished by means of smooth rollers, pneumatic tired rollers, tamping, or sheep's-foot rollers. The equipment selected shall be capable of compacting the soils to density requirements hereinafter specified.

When compaction is to be accomplished in places inaccessible to rolling, adequate mechanical tampers shall be used to obtain proper density.

(b) *Procedures*—Regardless of the method used, soils and rock-soil mixtures shall be compacted at moisture contents near those established by compaction and other laboratory tests as the optimum moisture content for the particular soil. A rather wide range on either side of the "optimum" moisture

content may be used for silty-clay soils. However, silty soils are extremely sensitive to changes of moisture, and must be compacted at moisture contents at or slightly below the optimum moisture content. Since most shales will disintegrate under freezing and thawing or wetting and drying conditions, shales must be compacted in at least a semi-disintegrated condition. Coarse-textured granular materials are best compacted by vibratory procedures. The use of water during the compaction of sands will provide a well compacted material without a detrimental dust condition.

(c) *Consolidation of road foundation*—All fills, base courses, and subbases shall be compacted in layers generally not greater than 9 in. loose depth. Each layer shall be compacted to minimum density not less than 95 percent expressed as a percentage of the maximum density and determined by "Laboratory Method of Test for the Compaction and Density of Soil" (AASHO T 99).

406—Testing and correcting the subbase

Immediately prior to placing concrete, the subbase shall be tested for conformity with the cross section shown on the plans by means of an approved template riding on the side forms. If necessary, material shall be removed or added, as required, to bring all portions of the subbase to the correct elevation. It shall then be thoroughly compacted and again tested with the template. Concrete shall not be placed on any portion of the subbase which has not been tested for correct elevation.

If the soil subbase is disturbed after acceptance, it shall be reshaped and compacted without additional compensation. Payment for compacting the subbase shall be included in the contract prices for subbase.

407—Placing subgrade paper

When specified, the subbase, after it has been shaped and compacted, shall be covered with subgrade paper. Adjacent strips of paper shall lap 4 in. and ends shall lap 12 in. After being placed on the subbase care should be exercised to keep the paper intact.

408—Wetting the subbase

Except where subgrade paper is required, the subbase shall be thoroughly wetted a sufficient time in advance of the placing of the concrete to insure that there will be no puddles or pockets of mud when the concrete is placed.

409—Grading stakes

Grading stakes shall be protected by the contractor. Stakes which become disturbed shall be reset at the contractor's expense.

410—Clearing right-of-way

Where fills are less than 10 ft high, all sod, underbrush, trees, tree stumps, and saplings shall be grubbed and entirely removed; when fills more than 10 ft high are to be made, the ground shall be cleared of all underbrush, and all trees and saplings shall be cut off not over 1 ft above the ground.

411—Solid rock

Solid rock (boulder of 1 cu yd or more in content, or ledges in their original bed) shall be removed 6 in. below grade and shall be paid for at the contract price per cu yd of solid rock.

412—Removal of unsuitable material

All unsuitable material shall be removed to a depth of at least 18 in. below the finished surface of the subbase. The contractor will be paid for the work at the prices bid for grading and materials used.

413—Excavated materials

Material excavated in cuts may be used in fills on the job provided the material conforms to the specifications for fill materials contained herein. Excess or unsuitable material shall be disposed of by the contractor at his expense.

Any excavation of approved soil below the surface of the subbase as shown on the plans shall be refilled with approved material compacted as provided herein for filling and compaction at the contractor's expense.

414—Borrowed material

Borrowed material shall be secured from pits indicated on the plans. If no such location is indicated or the quantity of the material at the pits is insufficient, the contractor shall furnish material conforming to the requirements of these specifications. Payment shall be made for such material at the price bid for borrow per cu yd.

415—Payment for grading

(a) The contract prices for grading shall include: the cost of excavating and filling; clearing and grubbing; excavating and removing unsuitable materials as required; shaping the subgrade to the prescribed lines and grades; sloping cuts, intersections, and approaches; rolling or tamping below the subgrade in the case of fills; and in connection with street construction, sloping and filling between the curb and sidewalk pavement, and grading and shaping of sidewalk spaces where shown on the plans. Payment shall be made at price bid per cu yd computed to the grades and slopes shown on the plans.

(b) If separate subbase and/or base, if required, is built of selected material during the grading operations, the payment for same shall be separate.

CHAPTER 5—FORMS

501—Material and dimensions

Side forms shall be made of metal having a thickness of not less than $\frac{7}{32}$ in. and shall have a depth equal to the specified edge thickness of the concrete. Building up of forms shall not be permitted. Flexible or curved forms of proper radius shall be used for curves of 100 ft radius or less. Forms shall not deflect more than $\frac{1}{4}$ in. when tested as a simple beam with a span of 10 ft and a load equal to that which the finishing machine or other construction

equipment will exert upon them. Forms 8 in. or more in height shall be at least 8 in. wide at the base; forms less than 8 in. in height shall have a base width at least equal to the height of the forms. The flange braces must extend outward on the base not less than two-thirds the height of the form. The forms shall be free from warp, bends, or kinks. The top of the form shall not vary from a 10-ft straightedge by more than $\frac{1}{8}$ in. at any point and the side of the form by more than $\frac{1}{4}$ in.

502—Form support

The soil foundation under the forms shall be compacted and cut to grade so that the forms, when set, shall be uniformly supported for their entire length and at the specified elevations. Such soil foundation found to be below established grade at the form line shall be filled to grade in lifts of $\frac{1}{2}$ in. or less for 18 in. on each side of the base of the form and thoroughly re-rolled or tamped. Imperfections and variations above grade shall be corrected by tamping or by cutting, as necessary.

503—Grade alignment

The alignment and grade elevations of the forms shall be checked and the necessary corrections made by the contractor immediately before placing the concrete. When any form has been disturbed or any subgrade thereunder has become unstable, the form shall be reset and rechecked.

504—Staking forms

Forms shall be staked with three or more pins for each 10-ft section. A pin shall be placed adjacent to each side of every joint if required. Form sections shall be tightly joined by a locked joint free from play or movement in any direction. Forms shall be cleaned and oiled prior to the placing of concrete.

505—Advance setting

Forms shall be set sufficiently in advance of the point where concrete is being placed as to permit proper progress and inspection of the work. At least 300 ft is desirable.

506—Removal of forms

Forms shall remain in place at least 12 hr after placing the concrete. If the air temperature is below 50 F at any time during the 12-hr period from the time the concrete is placed, the forms shall not be removed until 36 hr after placing of the concrete, unless high-early-strength concrete is being used.

Curing of the exposed concrete shall begin immediately upon removal of the forms as required in Section 1101(b).

CHAPTER 6—INSTALLATION OF JOINTS AND REINFORCEMENT

601—General

(a) All logitudinal and transverse joints shall conform to the details and positions shown on the plans.

(b) All transverse joints shall be constructed in line for the full width of the pavement.

(c) All joints shall be constructed true to line with their faces perpendicular to the surface of the pavement. Joints shall not vary more than ¼ in. from a true line or from their designated position. When plates are used, the joint shall be over the plate and the plate shall not vary more than ¼ in. from a true line.

(d) The surface of the pavement adjacent to all joints shall be finished to a straight line profile across all joints and shall be edged to a radius of ¼ in, or as otherwise shown on the plans. The surface across the joints shall be tested with a 10-ft straightedge as the joints are finished, and any irregularities in excess of ⅛ in. shall be corrected before the concrete has hardened.

(e) Keyways, when required, shall be accurately formed with templates of metal or wood. The gage, or thickness, of the material in the template shall be such that the full keyway as specified is formed.

602—Longitudinal joints

(a) *Metal keyway joints*—Longitudinal "metal strip" joints shall be formed by installing a metal parting strip to be left in place, meeting the requirements of Section 202(g). The metal strip shall be held securely in place, true to line and grade, by steel pins, meeting the requirements of Section 202(g), spaced at intervals that average not less than 3 ft and in no case more than 4 ft. The sections shall be securely fastened by lapping and pinning, by means of a slip joint, or other approved method. The contractor shall furnish an approved gage to ride on the side forms for checking the position of the parting strip before concrete is placed against it.

(b) *Weakened plane joints*—Longitudinal "dummy groove" or weakened plane joints shall be formed in the concrete by one of the following methods, as specified:

1. By making a groove in the plastic concrete with a suitable tooling device, and groove shall extend vertically downward from the surface to the required depth. The groove when formed by this method shall be ⅜ in. wide at the surface and ¼ in. wide at the bottom unless otherwise specified.

2. By sawing a groove in the concrete to the required minimum depth, and with a minimum width of cut of ¼ in. unless otherwise directed by the engineer; the required width may be secured by using two saw blades separated by spacers of proper thickness. The timing and order of sawing shall be directed by the engineer.

(c) *Construction joints*—Longitudinal construction joints (i.e. joints between slabs placed separately) shall be formed by the use of standard steel side forms with keyway, unless otherwise indicated in the plans. Provision shall be made for the installation of tie bars as noted in these specifications, or in the plans or in the special provisions.

(d) *Tie bars*—Tie bars not less than ½ in. in diameter and 30 in. long, unless otherwise specified, shall be placed across all longitudinal joints. Tie

bars shall be placed at right angles to the centerline and shall be spaced at intervals of 30 in. unless otherwise shown on the plans. They should be held in a position approximately parallel to the transverse axis of the pavement and midway between the top and bottom surfaces of the slab. Tie bars may be bent at right angles against the form at longitudinal construction joints if permitted, unless threaded bolt or other assembled tie bars are required. Joint hook bolts conforming to the provisions of Section 202(h)4 may be used. The hook bolt and coupling should be provided with an approved fastener for attachment to the pavement form to maintain them in corrected position during concreting and subsequent removal of the forms.

603—Expansion joints at structures

Expansion joints shall be formed about all structures and features projecting through, into or against the pavement. Unless otherwise indicated on the plans, such joints shall be not less than ¼ in. thick and shall be of the premolded type.

604—Transverse expansion joints

Transverse expansion joints shall be constructed at right angles to the centerline of the pavement, unless otherwise required, and shall extend the full width of the pavement.

(a) *Premolded expansion joints*—Transverse expansion joints shall be formed by securely staking in place an approved load transfer device (Section 608) consisting of an assembly of dowels, supporting and spacing means, and premolded joint filler. The filler shall extend the full width of the pavement. The bottom edge of the filler shall extend downward to or slightly below the bottom of the slab, and the top edge, unless otherwise prescribed, shall be held about ½ in. below the surface of the pavement to allow the finishing operations to proceed continuously. The top edge of the filler shall be protected, while the concrete is being placed, by a metal channel cap of at least ⅛-in. material, having flanges not less than 1½ in. deep.

The joint assembly shall be protected against damage until it is installed in the work. Joints damaged during transportation, or by careless handling, or while in storage shall be replaced or repaired by the contractor. Damaged or repaired joints shall not be used until they have been approved by the engineer.

(b) *Premolded joint filler*—The designated premolded joint filler shall be appropriately punched to the exact diameter and at the location of the dowels. It shall be furnished in lengths equal to one-half the designated width of the slab. Where more than one section is used in a joint, they shall be securely placed or slipped together. The premolded joint filler shall be placed on the side of the installing bar nearest the mixer. The bottom edge of the filler shall extend downward to or slightly below the bottom of the slab, and the top edge, unless otherwise prescribed, shall be held about ½ in. below the surface of the pavement in order to allow the finishing operations to proceed continuously. The top edge of the filler shall be protected while the concrete is being placed, by a metal channel cap of at least 10 gage material, having

flanges not less than 1½ in. deep. The installing device may be designed with this cap self-contained.

605—Transverse contraction joints

(a) *Metal strip type*—Transverse "metal strip" contraction joints shall be formed during the placing of the concrete by installing a metal parting strip to be left in place. This strip shall conform to the requirements for type, gage, shape, and dimensions shown on the plans or indicated in the special provisions and shall be securely staked in place. This strip shall be temporarily capped with a metal channel cap. The dowel assembly, Section 608, and the method of placing it shall be as indicated for transverse expansion joints, except that sleeves or caps on dowels will not be required. Concrete shall be deposited as for transverse expansion joints. After the longitudinal floating has been completed but before the surface is finished, the metal cap shall be removed and the concrete edged, and the joint sealed as required for expansion joints.

(b) *Weakened plane joints*—Transverse "dummy groove" or weakened plane joints shall be constructed in the same manner as provided for weakened plane joints for longitudinal joints, Section 602(b), except that such joints shall be provided with slip dowels or other load transfer devices of the type, size, and arrangement shown on the plans. The spacing and depth shall be as indicated on the plans.

606—Weakened plane warping joints

Weakened plane warping joints, when specified, shall be constructed in the locations shown on the plans and in the same manner as the weakened plane contraction joints except that the load transfer devices shall be omitted and the pavement reinforcing shall extend through the joint.

607—Transverse construction joints

Unless other prescribed joints occur at the same points, transverse construction joints shall be made at the end of each day's run or where interruption of more than 30 min occurs in the concreting operations.

Transverse construction joints shall be formed by staking in place a bulkhead of timber approximately 4 in. wide and of the same depth as the thickness of the concrete pavement slab to be laid.

Transverse construction joints shall not be formed so as to make a slab less than 10 ft in length, and if sufficient concrete is not mixed to form a slab at least 10 ft in length, the joints shall be formed at the preceeding joint location and the excess concrete disposed of as directed by the engineer.

The spacing of subsequent transverse joints shall be measured from the transverse construction joint last placed.

608—Load transfer devices

(a) *Dowel bars*—Dowel bars at least ¾ in. in diameter, 16 in. long and spaced at 12 in. center-to-center, or other approved type of load transfer device, shall be placed across all transverse joints, except warping joints and

joints in base, in the manner shown on the plans. Dowels shall be placed at the middle of the slab depth and held rigidly in proper horizontal and vertical alignment by an approved dowel assembly device [Section 604(a)] to be left permanently in place. A tolerance of not more than $\frac{1}{8}$ in. in 12 in. from correct alignment, either vertical or horizontal, will be permitted.

(b) *Dowel coating*—The free or unbonded end of each dowel shall be painted over one-half its length with one coat of lead or asphaltic paint. When the paint has dried, the free end of each bar shall be thoroughly coated with a thin brush coat of MC-2 immediately before it is placed in position. The free ends of the dowel bars for expansion joints shall be provided with a metal dowel sleeve conforming to the dimensions shown on the plans, or as approved by the engineer.

609—Installation of joints

(a) If the paving mixer is operated from the shoulder, the joints shall be set immediately after the final testing of the subgrade. If the paving mixer is operated from the subgrade, the joints shall be set immediately after it moves forward, so as to permit as much time as possible for proper installation.

(b) The assembled joint shall be put in place on the prepared subgrade. Transverse joint assemblies shall be placed at right angles to the center line of the pavement. The top of the joint assembly shall be set at the proper distance below the pavement surface and the elevation checked by a properly designed template. On widened curves, the longitudinal center joint shall be so placed that it will be, as nearly as practicable, equidistant from the edges of the slab. The joint shall be set to the required line and grade and shall be securely held in the required position by stakes or an approved installing device [Section 604(a)] or both during the placing and finishing of the concrete. Joints shall be installed so that the concrete pressure will not disturb their alignment. The joints shall be vertical and no joint shall deviate more than $\frac{1}{4}$ in. in horizontal alignment either way from a straight line. If joints are constructed in sections, there shall be no offsets between adjacent units. Dowel bars shall be checked for exact position and alignment as soon as the joint is staked in place on the subgrade, and the joint shall be tested to determine whether it is firmly supported. Any joint not firmly supported shall be reset.

610—Joint sealing

(a) The top of expansion joints and all edged joints shall be sealed with the specified sealing material before traffic is permitted on the pavement. The joint opening shall be thoroughly cleaned of all foreign matter before the sealing material is placed. All contact faces of the joint shall be cleaned with a wire brush to remove loose material, and shall be surface dried when hot-poured sealing material is used.

(b) The sealing material shall be poured or placed into the joint opening to conform to the details shown on the plans or as directed by the engineer. The pouring shall be done in such a manner that the material will not be

spilled on the exposed surfaces of the concrete. Any excess material on the surface of the concrete pavement shall be removed immediately and the pavement surface cleaned.

(c) When required to prevent tackiness or pickup under traffic, the exposed surfaces of the sealer shall be dusted with an approved material. Other methods of preventing pickup under traffic may be used when approved by the engineer.

(d) Poured joint sealing materials shall not be placed when the air temperature in the shade is less than 50 F except by the approval of the engineer.

611—Placing reinforcement

(a) Steel reinforcement shall consist of welded fabric or bar mats in accordance with Section 202(h) of these specifications. Reinforcing steel shall be free from dirt, scale, or other foreign matter, and rust of such degree of development as to impair bond of the steel with the concrete. The width of fabric sheets or bar mats shall be such that, when properly placed in the work, the extreme longitudinal members of the sheet or mat will be located not less than 3 in. nor more than 6 in. from the edges of the slab. Except for warping joints, the length of fabric sheets or bar mats shall be such that, when properly placed in the work, the reinforcement will clear all transverse joints by not less than 2 in. nor more than 4 in. as measured from the center of the joint to the tip ends of the longitudinal members of the sheet or mat.

(b) When bar assemblies are shown on the plans, the reinforcing bars shall be firmly fastened together at all intersections. Adjacent ends shall lap not less than 20 diameters.

(c) Where bars are fabricated into mat form by positive welding at all intersections, the lap may be of such length as will permit cross bars to overlap each other by at least 2 in. The same requirements apply to end laps and side laps.

(d) Steel fabric sheets shall be lapped a distance at least equal to the spacing between wires.

(e) Reinforced concrete shall be placed in two operations. The initial layer shall be uniformly struck off at a depth not less than 2 in. nor more than one-third the total depth of slab below the proposed surface of the pavement, and the reinforcement placed thereon. The concrete shall be struck off to the entire width of the pour and a sufficient length to permit the sheet or mat of reinforcement to be laid full length on the concrete in its final position without further manipulation of the reinforcement. The balance of the required concrete shall be placed on the reinforcement. Displacement of the reinforcement during concrete operations shall be prevented.

CHAPTER 7—CONCRETE PROPERTIES AND PROPORTIONS OF MATERIALS

701—Basis of proportions

The proportions of water, cement, and aggregates shall be in accordance with proportions based on design for strength (Section 703) or in accordance

with proportions based on uniform cement factor (Section 704) as specified in the special provisions.

702—Air content

When concrete is made with an air-entraining cement or air-entraining admixture, the percentage of air shall be maintained between 4 and 7 percent as determined by methods stated under Section 306 of these specifications. Where frost action is not a factor and the entrained air is used for the purpose of improving the properties of the fresh concrete, these limits may be reduced to 2 and 5 percent.

703—Proportions based on design for minimum strength

(a) *Proportions*—The proportions of cement, fine aggregate, coarse aggregate, and water to be used in the mix shall be determined by the engineer, within the limits of Sections 703(b), 703(c), and 703(d), by laboratory tests of the flexural strength of concrete made with aggregates from the same sources and of the same gradings as will be employed in the work and that portland cement which is found to produce the lowest strength concrete of any acceptable cement which will be used in the work.

(b) *Consistency*—The mixture determined upon shall produce workable concrete having a slump of 1½ to 3 in. for unvibrated concrete, or ½ to 1½ in. for vibrated concrete.

(c) *Strength*

1. The average compressive strength, for use with the allowable stresses for design of dowels and tie bars as recommended in "Recommended Practice for Design of Concrete Pavements" (ACI 325), shall be not less than 4000 psi at 28 days, when specimens are molded and tested in accordance with Section 304.

2. The average flexural strength of concrete as determined by the laboratory test specified in Section 302 should be not less than 650 psi at 28 days. If known characteristics of the available cements and aggregates or preliminary tests in accordance with Section 302 indicate difficulty in attaining this strength economically, an average flexural strength of 600 psi may be specified provided the slab thickness is designed accordingly. In the case of the 650 psi strength the adequacy of the laboratory design is confirmed if at least 80 percent of all flexural strength test results of specimens molded in accordance with Section 303(a) and cured in accordance with Sections 7(a) and (b) of ASTM C 31 referred to therein, shall be equal to or greater than 600 psi and the average of any four consecutive tests shall be equal to or greater than 600 psi. In the case of the 600 psi strength the adequacy of the laboratory design is confirmed if 80 percent of all flexural strength test results of specimens molded in accordance with Section 303(a) and cured in accordance with Sections 7(a) and (b) of ASTM C 31 referred to therein, shall be equal to or greater than 550 psi, and the average of any four consecutive tests shall be equal to or greater than 550 psi.

3. The strengths at 7 and 28 days shall be established with four cement factors prior to starting the work. These will serve as a basis for adjusting the mix when this need is indicated by 7-day tests, made during the progress of the work on job-made, laboratory-cured specimens, later checked by 28-day tests on specimens similarly made and cured.

(d) *Durability requirements*—In localities where the concrete will be subjected to severe freezing and thawing conditions, air-entrained concrete shall be used, and the resulting proportions shall be such that the mixing water, including free surface moisture on the aggregates but exclusive of moisture absorbed by the aggregates, shall not exceed 6 gal. per sack of cement. In no case shall the mixing water exceed 6½ gal. per sack of cement for any individual batch. The cement content shall be not less than 5 sacks per cu yd.

(e) *Information for bidders*—Unless otherwise specified in the special provisions the following requirements shall govern the contractual relations concerning proportions based on design for minimum strength:

1. Upon request, the engineer will furnish prospective bidders with information as to the proportions by weight required for aggregates from established sources available for use on the project. This information will also include the grading of the aggregates used in determining these proportions.

2. Promptly after receipt of notice of award of the contract, the contractor shall furnish the engineer with the location or locations of the source or sources of aggregates which he proposes to use. The proportions will be designated by the engineer. Except as hereinafter provided, the designated proportions shall govern as long as materials are furnished from the sources designated and as long as they continue to meet the requirements specified.

3. If, during the progress of the work, the contractor proposes to use aggregates from approved sources other than those originally designated, the engineer will designate the new proportions to be used.

4. If satisfactory plasticity and workability are not secured using the proportions and aggregates originally designated, the engineer may alter such proportions. If such alterations change the designated cement factor originally fixed by 2 percent or less, no adjustment in the amount paid the contractor shall be made. If such alterations change the designated cement factor by more than 2 percent, the source and quality of the aggregates remaining the same, payment shall be adjusted for or against the contractor in whatever amount the total cost of materials, f.o.b. contractor's material yard, has been increased or decreased by more than 2 percent. The calculation of the amount of such increase or decrease shall be based upon the designated cement factor and not on count of bags of cement used or of the batches where bulk cement is used.

704—Proportions based on fixed cement content

When the concrete having a fixed cement content is specified, the quantities shall be those given in Table 704 properly adjusted to compensate for differences in specific gravities. Upon approval by the engineer, these

TABLE 704—SUGGESTED TRIAL PROPORTIONS FOR CONCRETE OF SPECIFIED CEMENT CONTENT AND SLUMP*

Type of concrete†	Type of coarse aggregate	Mixture proportions per 94-lb bag of cement‡			
		Water, gal.	Aggregates, lb		
			Fine§	Coarse**	
				Small	Large
Plain	Round gravel	5.0	185	145	200
Plain	Crushed gravel or stone	5.5	200	135	200
Plain	Crushed slag	6.0	220	105	155
Air-entrained	Round gravel	4.5	160	145	220
Air-entrained	Crushed gravel or stone	5.0	180	135	200
Air-entrained	Crushed slag	5.5	195	105	155

*Proportions intended to produce concrete containing 6.0 bags of cement per cu yd with slump of 1½ to 3 in., suitable for normal machine placement. When vibration is used, slump may be reduced to about ½ to 1½ in. and batch quantities adjusted accordingly to maintain same yield and cement factor.

†Air content assumed to be 1 percent for plain mixes and 5.5 percent for air-entrained mixes.

‡Aggregate weights based on assumed bulk specific gravity, saturated-surface-dry, of 2.65 for sand, gravel, and stone and 2.25 for slag. For other specific gravities, aggregate weights should be adjusted in direct proportion to the specific gravity. Aggregate weights and quantity of added mixing water must be adjusted to allow for free moisture on aggregates.

§Fine aggregate assumed to be well graded natural sand of average fineness (fineness modulus about 2.6 to 2.9).

**Coarse aggregates assumed to be well-graded from 1½ in. to No. 4 or 2 in. to No. 4, furnished in two sizes separated on the ¾-in. or 1-in. sieve, respectively, and used in the proportion of approximately 40 percent of the small and 60 percent of the large size.

compensated quantities of materials shall be further adjusted by the engineer at any time to attain the desired degree of workability, the specified slump, and the specified cement content. Batch weights thus determined, shall be corrected to take into account the moisture condition of the aggregates as used.

CHAPTER 8—HIGH-EARLY-STRENGTH CONCRETE

801—Methods of production

High-early-strength concrete shall be produced by one of the following methods, or any combination thereof, as specified in the special provisions.

(a) By the use of high-early-strength portland cement Type III or IIIA in lieu of normal portland cement (Type I or IA, or Type II or IIA).

(b) By the use of additional portland cement (Type I or IA, or Type II or IIA), in which case the total amount of cement shall not exceed 8 bags of cement per cu yd of concrete.

(c) By the use of calcium chloride as one of the ingredients of the concrete in an amount between 1 and 2 lb of calcium chloride per bag of cement. Calcium chloride shall be added in solution. It is convenient to so proportion the solution that 1 qt contains 1 lb of calcium chloride.

802—Requirements

High-early-strength concrete shall meet all the requirements of the specifications for portland cement concrete.

CHAPTER 9—MIXING

901—Measurement and handling of materials

(a) Standard size sacks of cement as packed by the manufacturer shall be considered to weigh 94 lb net. Either package or bulk cement may be used

but not both. Bulk cement and cement from fractional sacks shall be weighed. Cement shall be protected from the weather and against loss in handling or in transit.

(b) When package cement is used, the cement shall be emptied from the sacks into the batch immediately prior to mixing. When bulk cement is used, satisfactory methods of handling and weighing shall be employed. A separate hopper, so designed and operated that the quantity of cement for each batch will be maintained in a separate container and not come in direct contact with the aggregates, shall be used.

(c) Aggregates shall be weighed.

(d) The scales for weighing cement and aggregates shall be accurate within ½ of 1 percent throughout the range of use. Operation shall be within a maximum allowable error of 1 percent for cement and 2 percent for aggregates.

(e) Mixing water may be measured by weight or by volume. The water-measuring device shall be such that the operation shall be accurate to within 1 percent. When wash water is used as a portion of the mixing water for succeeding batches, it shall be measured according to this requirement.

902—Mixing at site

(a) The concrete shall be mixed in a batch mixer. When a drum mixer is used, it shall conform to the requirements of the concrete mixer standards of the Mixer Manufacturer's Bureau of the Associated General Contractors of America. If another type of mixer is used, it shall be of a type meeting the approval of the engineer. The mixer shall be capable of combining the aggregates, cement, and water into a thoroughly mixed and uniform mass within the specified time, and of discharging the mixture without segregation. Each batch of concrete shall be mixed for 1 min or more after all materials exclusive of the mixing water, are in the mixer drum. The batch shall be so charged into the mixer that some water will enter in advance of the cement and aggregate, and will continue to flow for a period which may extend to the end of the first one-third of the specified mixing time. The mixer shall rotate at the rate recommended by its manufacturer. Concrete shall be mixed only in quantities required for current use; any concrete which has set so that it cannot be properly placed shall not be used. Retempering of concrete which has partially set, by mixing with additional water, will not be permitted.

(b) The mixer shall be equipped with a suitable charging hopper, water storage, and a water-measuring device. Controls shall be so arranged that the water can be started only while the mixer is being charged and so as to lock automatically the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged after each batch has been mixed the required time. Suitable equipment for discharging and spreading the concrete on the subgrade shall be provided. The mixer shall be cleaned at suitable intervals.

The pick-up and throw-over blades in the drum shall be replaced when they have lost 10 percent of their depth. The volume of the mixed material per batch shall not exceed the manufacturer's rated capacity of the mixer. The manufacturer shall install a plate upon the mixer stating the rated capacity and the recommended revolutions per minute.

903—Ready-mixed concrete

Ready-mixed concrete shall be mixed and transported in accordance with the "Specifications for Ready-Mixed Concrete" (ASTM C 94). When ready-mixed concrete is used in the construction of portland cement concrete pavements, a suitable spreader shall be provided.

CHAPTER 10—PLACING AND FINISHING CONCRETE

1001—General

(a) Concrete shall be distributed to such depth, above grade, that when consolidated and finished, the specified slab thickness will be obtained.

(b) The concrete shall be deposited on the subgrade in such a manner as to require as little rehandling as possible, preferably by a mechanical spreader. It shall be thoroughly consolidated against and along the faces of all forms with a vibrator inserted in the concrete. Necessary hand spreading shall be done with shovels, not with rakes. Workmen shall not be allowed to walk in the concrete with boots or shoes covered with earth or other foreign substances.

(c) Concrete shall be placed only on subgrade which has been prepared as specified and approved. At all times during operations, adequate subgrade shall have been prepared ahead of the mixer; at least 300 ft is desirable. Concrete shall not be placed on a frozen subgrade. No concrete shall be placed around manholes or other structures until they have been brought to the required grade and alignment.

(d) Pavement 30 ft or less in width may be constructed either to its full width in a single construction operation or in lanes, unless one or the other method is expressly stipulated on the plans. Pavement more than 30 ft in width shall not be constructed in a single operation. When pavement is constructed in separate lanes, the longitudinal construction joints shall not deviate from the true line shown on the plans by more than ½ in. at any point.

(e) Retempering concrete by adding water will not be permitted.

(f) If ready-mixed concrete is used a suitable spreader shall be provided.

(g) When natural light is insufficient for proper work, adequate artificial lighting shall be provided.

1002—Placing and finishing concrete at joints

(a) Concrete shall be deposited on the subgrade as near to the expansion and contraction joints as possible without disturbing them. It shall then be shoveled against both sides of the joint simultaneously, maintaining equal pressure on both sides. It shall be deposited to a height of approximately 2

in. more than the depth of the joint, care being taken that it is worked under the load transfer devices. The concrete shall not be dumped from the discharge bucket of the mixer directly on top of the load transfer devices. In placing the concrete against expansion and contraction joints and in operating a vibrator adjacent to them, workmen shall avoid stepping upon or disturbing in any way the joints or load transfer devices, either before or after they are covered with concrete.

(b) The concrete adjacent to the joints shall be compacted with a vibrator inserted in the concrete. The vibrator shall be worked along the entire length and on both sides of the joint. The vibrator shall not come in contact with the joint, the load transfer devices, the forms, or the subgrade. If any of the dowel bars are displaced, they shall be realigned before the finishing machine passes over them.

(c) After the concrete has been vibrated, the finishing machine, as specified in Section 1003, shall be moved forward until the front screed is approximately 8 in from the joint. Segregated coarse aggregate shall be removed from both sides of and off the joint. The screed shall be lifted and brought directly above the joint, set upon it, and the forward motion of the finishing machine shall be resumed. When the second screed is close enough to permit the excess mortar in front of it to flow over the joint, it shall be lifted and carried over the joint. Thereafter, the finishing machine may be run over the joint without lifting the screeds, provided there is no segregated coarse aggregate immediately between the joint and the screed or on top of the joint.

(d) After the concrete has been placed on both sides of the joint and struck off, the installing bar or channel cap shall be slowly and carefully withdrawn. After the installing bar or channel cap is completely withdrawn, the concrete shall be carefully spaded and additional freshly mixed concrete worked into any depressions left by the removal of the installing bar. The installing bar shall be cleaned and reoiled prior to reuse.

Immediately after all finishing operations have been completed and before the concrete has taken its initial set, it shall be edged adjacent to all expansion and contraction joints. Care shall be used to remove any concrete which may be over the premolded joint material. The edging tool shall be so manipulated that a well-defined, continuous radius is produced and a smooth, dense mortar finish obtained.

After the removal of the side forms, the ends of premolded transverse joints at the edges of the pavement shall be carefully opened for the entire depth of the slab, and any concrete that has been deposited over the end closure shall be removed, care being taken not to injure the ends of the joint. After the curing period and before the pavement is opened to traffic, all joints shall be sealed, leaving a nearly uniform strip of an approved sealer material.

1003—Finishing methods

(a) *Machine finishing*—The concrete as soon as placed shall be struck off and screeded by an approved finishing machine to the crown and cross

section shown on the plans and to an elevation slightly above grade so that when properly consolidated and finished the surface of the pavement will be at the exact grade elevation indicated on the plans and free from porous places. The finishing machine shall be of the screeding and troweling type, equipped with two independently operated screeds, designed and operated to strike off the concrete. The machine shall go over each area of pavement as many times and at such intervals as is necessary to give the proper compaction and to leave a surface of uniform texture, true to grade and crown. Excessive operation over a given area shall be avoided. The tops of the forms shall be kept clean by an effective device attached to the machine, and the travel of the machine on the forms shall be maintained true without lift, wobbling, or other variation tending to affect the precision finish. The finishing machine shall be of ample strength to withstand severe use and shall be fully and accurately adjustable for loss of crown or other derangement due to wear.

During the first pass of the finishing machine a uniform ridge of concrete shall be maintained ahead of the front screed for its entire length. Except when making a construction joint, the finishing machine shall not be operated beyond that point where the above described surplus can be maintained ahead of the front screed.

(b) *Vibrated finishing*—When vibrated finishing is required, the finishing machine shall be equipped for applying high frequency vibration to the upper surface of the concrete. The vibratory units shall be synchronized and shall operate at a frequency of not less than 3500 cycles per min.

(c) *Floating*—After the concrete has been struck off and consolidated, it shall be further smoothed and consolidated by means of a longitudinal float of a suitable design approved by the engineer. Care should be exercised to start the floating operation at the proper time. In this operation, the longitudinal float shall be worked with a reciprocating motion, while held in a floating position parallel to the road center line and passed gradually from one side of the pavement to the other. Movements ahead along the center line of the road shall be in successive advances of not more than one-half the length of the float.

(d) *Straightedging*—After the longitudinal floating has been completed and the excess water removed, but while the concrete is still plastic, the contractor shall test the slab surface for trueness with a 10-ft straightedge. The straightedge shall be held in successive positions parallel to the road center line in contact with the surface and the whole area gone over from one side of the slab to the other. Advance along the road shall be in successive stages of not more than one-half length of the straightedge. Any depressions found shall be filled immediately with freshly mixed concrete, struck off, consolidated and refinished. High areas shall be cut down and refinished. The straightedge testing and refloating shall continue until the entire surface is found to be free from observable departures from the straightedge and the slab has the required grade and crown.

(e) *Belting*—After straightedging when most of the water sheen has disappeared and just before the concrete becomes non-plastic, the surface shall be belted with a two-ply canvas belt not less than 8 in. wide and at least 3 ft longer than the width of the slab. Hand belts shall have suitable handles to permit controlled uniform manipulation. The belt shall be operated with short strokes transverse to the road center line and with a rapid advance parallel to the center line.

(f) *Brooming or burlap drag*—After belting and as soon as surplus water has risen to the surface, the pavement shall be given a broom or burlap drag finish as directed.

If a broom finish is given, it shall be accomplished with an approved steel or fiber broom, not less than 18 in. wide. The broom shall be pulled gently over the surface of the pavement from edge to edge. Adjacent strokes shall be slightly overlapped. Brooming shall be perpendicular to the center line of the pavement and so executed that the corrugations thus produced will be uniform in character and width, and not more than $\frac{1}{8}$ in. deep. The broomed surface shall be free from porous spots, irregularities, depressions, and small pockets or rough spots such as may be caused by accidentally disturbing particles of coarse aggregate, embedded near the surface. Brooming will not be required for concrete bases.

If a burlap drag finish is given, a burlap belt shall be dragged in the longitudinal direction in one continuous motion. The use of a burlap belt which produces corrugations more than $\frac{1}{8}$ in. deep will not be permitted. The burlap shall be cleaned by washing or rinsing as often as necessary to prevent hardened concrete from accumulating therein and consequent scarring of the pavement surface.

(g) *Edging*—After final finishing of the concrete has been completed, but before the concrete has taken its initial set, the edges of the slab shall be carefully finished with an edger of the specified radius.

(h) *Final surface test*—The contractor will be held responsible for the correct alignment, grade, and contour specified. Any areas higher than $\frac{1}{8}$ in. for concrete pavements and higher than $\frac{1}{4}$ in. for concrete bases but not higher than $\frac{3}{8}$ in. above the correct surface as shown by the 10-ft straightedge, shall be ground to the required surface by the contractor at his own expense. When deviation exceeds the foregoing limits, the pavement slab shall be removed and replaced by the contractor at his own expense, as directed by the engineer.

CHAPTER 11—CURING AND PROTECTION OF CONCRETE

1101—Curing

(a) *Preliminary curing period*—The concrete shall be covered with two thicknesses of damp burlap, cotton mats or other approved material of highly absorptive quality, as soon as possible without damaging the finish. The material shall be kept damp by spraying and shall remain in place for at least 12 hr.

(b) *Final curing*—For completion of the curing the mats used for the preliminary curing period may be left in place and kept saturated for 72 hr, or may be removed at the end of the preliminary curing period and the concrete surface covered with paper, impermeable sheets, or liquid membrane-forming curing compound meeting the requirements of these specifications.

(c) *Other methods*—Other methods of curing may be used if approved by the engineer.

CHAPTER 12—CONCRETE BASES

1201—Transverse weakened plane joints—concrete base

Concrete bases shall be provided with expansion and/or contraction joints, and shall be constructed in accordance with provisions of these specifications for constructing similar joints in concrete surface courses.

Note: For concrete bases it is especially important to reduce the interval between joints by means of closely spaced contraction joints so as to control the amount of joint opening (which may extend through the surface course) due to shrinkage and contraction. Experience indicates that the best results are obtained when the spacing of such joints does not exceed 20 ft; preferably the spacing should not exceed 15 ft. Load transfer devices are not required in concrete base construction.

CHAPTER 13—COLD AND HOT WEATHER CONCRETING

1301—Cold weather concreting

(a) Except by specific written authorization, concrete placing shall cease when the descending air temperature in the shade and away from artificial heat falls below 40 F. It shall not be resumed until the ascending air temperature in the shade and away from artificial heat rises to 40 F.

(b) When concreting is permitted during cold weather the temperature of the mix shall be not less than 60 F nor more than 80 F at the time of placing in the forms.* The aggregates or water or both may be heated. The aggregates may be heated by steam or dry heat prior to being placed in the mixer. The water shall not be hotter than 175 F; aggregates shall not be used which are hotter than 150 F.

(c) When concrete is being placed during cold weather and the air temperatures may be expected to drop below 35 F, a supply of straw, hay, grass, or other suitable blanketing material shall be provided along the line of the work. At any other time when the air temperature may be expected to reach the freezing point during the day or night, the material so provided shall be spread over the concrete to a sufficient depth to allow curing of the concrete.† Such protection shall be maintained for at least 5 days. If required by the engineer, concrete less than 24 hr old shall also be covered by approved canvas or similar enclosures and devices capable of maintaining the temperature within the concrete at 50 F or higher. Concrete injured by frost action shall be removed and replaced at the contractor's expense.

*See Table 1, "Recommended Practice for Winter Concreting (ACI 604-56)," for recommended concrete temperatures.

†See Tables 2b and 2c, "Recommended Practice for Winter Concreting (ACI 604-56)" for insulation requirements for concrete slabs on ground.

1302—Hot weather concreting*

(a) Except by written authorization, concrete placing shall cease if the temperature of the plastic concrete cannot be maintained at 90 F or lower.

(b) To facilitate the placement of concrete in hot weather, the aggregates or water or both may be cooled. Crushed ice is acceptable for cooling water. Spraying aggregates will result in cooling due to evaporation. The temperature of cement at time of batching should not exceed 160 F.

CHAPTER 14—MISCELLANEOUS

1401—Pavement thickness

(a) The thickness of the pavement shall be determined by measuring the lengths of drilled concrete cores according to the "Method of Measuring Length of Drilled Concrete Cores" (ASTM C 174). At such points as the engineer may select, each 1000 linear ft of pavement, two or more cores shall be taken and measured. The average thickness of each mile of slab, or fractional mile, will be determined from these measurements.

(b) Pavement of which the average thickness is not more than ⅛ in. less than the thickness required by the typical cross section shown on the plans, will be accepted and paid for at the contract price.

(c) The unit price for pavement, the average thickness of which is less than the thickness shown on the plans by more than ⅛ in. but less than ½ in. shall be adjusted. This price shall bear the same ratio to the contract unit price as the square of the average thickness of the slab bears to the square of the thickness specified on the plans.

(d) Payment will not be made for pavement which is found deficient in thickness by ½ in. or more. Such pavement shall be removed and replaced by the contractor with pavement of the specified thickness at his expense. When the measurement of any core indicates that the slab is deficient in thickness by ½ in. or more, determination shall be made of the thickness of transverse sections of the slab at 25-ft intervals set off along the center line of the road in each direction from the affected location until a transverse section of the slab is found which is not deficient in thickness by as much as ½ in. The area of pavement for which no payment will be made shall be the product of the width of pavement multiplied by the distance along the center line of the road between the transverse sections found not deficient in thickness by as much as ½ in. If the contractor is not satisfied, he may request additional cores and measurements. Such measurement shall be made at intervals or not less than 200 ft. The cost of additional cores and measurements shall be deducted from any sums due the contractor unless the measurements indicate that the slab within the area in question is of specified thickness.

(e) No additional payment over the unit contract price bid will be made for pavement found to be thicker than the specified amount.

*Special precautions as directed by the engineer shall be required for all high-early-strength concrete placed during hot weather.

1402—Protection of finished pavement

(a) The contractor shall erect and maintain suitable barricades and, when required in the special provisions, shall employ watchmen to exclude traffic from the newly constructed pavement until open for use. These barriers shall be so arranged as not to interfere with public traffic on any lane intended to be kept open, and necessary signs and lights shall be maintained by the contractor clearly indicating any lanes open to the public. Where, as shown on the plans or indicated in the special provisions, it is necessary to provide for traffic across the pavement, the contractor shall at his own expense, construct suitable and substantial crossings to bridge over the concrete.

(b) Any part of the pavement damaged by traffic or other causes prior to its final acceptance shall be repaired or replaced by and at the expense of the contractor in a manner satisfactory to the engineer. The contractor shall protect the pavement against both public traffic and traffic of his employees and agents.

1403—Public use of thoroughfare

(a) Normal, unimpeded use of the thoroughfare of which the proposed pavement is to be a unit, is of value to the public. It is, therefore, mutually understood, that for the sections of the thoroughfare identified on the plans as requiring special traffic handling and for the distances stated thereon, surfaced road lanes as indicated shall be made available by the contractor for unimpeded public traffic at all times, and maintained in proper condition throughout the construction period. These lanes shall be of the clear widths stated on the plans, and shall be kept entirely free from encroachment by equipment of the contractor, by workmen or employees of the contractor or by storage or transportation of materials intended for the work.

(b) The scheme and sequence of construction of the several lanes, slabs and sections of pavement, including the sequence of the shifting of public lanes during progress of construction, shall be as given on the plans or as described in the special provisions.

(c) Where the edge of any stipulated public traffic lane is contiguous to an edge of the slab or lane being placed, the contractor shall provide, erect, and subsequently remove, a substantial temporary guard fence, as shown on the plans along the prescribed dividing line, which shall be maintained there until the slab is opened to traffic. The contractor's plan of operation shall be such as to obviate any need for encroachment on the public traffic lane or lanes. Where so shown on the plans, special lanes for the contractor's trucks and similar vehicles shall be provided, separate from and not interfering with the prescribed traffic lanes. Where the clearance between public traffic lanes and the contractor's operating equipment is restricted, special delivering equipment may be necessary, designed to deliver and depart within the width of the slab actually being placed without encroaching any public lane.

(d) Except where a special bid price for "traffic handling" is required in the proposal and in the special provisions, all cost of handling and pro-

tecting traffic, of special equipment, of temporary road surfacing and its maintenance, of temporary guard fences and of other things to be provided or to be done under this paragraph, shall be at the expense of the contractor.

1404—Opening to traffic

(a) Traffic will ordinarily be excluded from the newly constructed pavement for 7 days after the concrete is placed and may be excluded for a longer period if flexural tests indicate its advisability. Job-cured flexural test specimens, prepared at regular intervals from the concrete as it comes from the mixer may be employed as a means of determining the time of opening the pavement to traffic. These beams shall be made and tested in accordance with Section 303. When tests of these specimens indicate that the corresponding pavement has attained a modulus of rupture of at least 500 psi, the pavement shall be cleaned, the joints filled and trimmed, and the pavement opened to traffic. The joint or line of separation between adjacent strips or slabs of concrete, when the pavement is constructed in strips or slabs, shall be cleaned and filled with an approved sealing material.

(b) Until the pavement is opened to public traffic as provided above, construction equipment shall not be permitted on the new pavement unless special precautions are taken to protect the pavement. Such use and precautions shall be subject to the approval of the engineer.

Discussion of this report should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

ACI Standard

Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)*

Reported by ACI Committee 711

LEO M. LEGATSKI
Chairman

RALPH W. ADAMS
J. H. APPLETON
FRED C. BAMMAN
MERWIN G. BEAVERS
FRANK BROMILOW
S. J. CHAMBERLIN
WILLIAM C. GREEN
HAROLD B. HEMB
GEORGE E. LARGE
F. N. MENEFEE
C. F. MOORE
O. NEIL OLSON
GAYLE B. PRICE
ROBERT A. ROSSI
R. N. VAKIL

SYNOPSIS

Minimum standard requirements for single units or multiple element assemblies, to be used in conjunction with ACI 318-56. Covers materials; design principles; manufacture including curing and handling; testing of completed units; installation plans; and special provisions for holes and openings in members. Design chapter treats such problems as dimensions, allowable deflection, structural concrete topping, reinforcement anchorage and location, and use of lightweight concrete.

CHAPTER 1—GENERAL

101—Scope and limits

(*a*) These minimum standard requirements for precast concrete floor and roof units are to be used in conjunction with the "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

(*b*) Precast floor and roof units which are manufactured as single units, or multiple element assemblies, are to be designed in accordance with standard reinforced concrete theory and as set forth in ACI 318-56 except that the requirements for concrete protection for reinforcement, bar spacing, bond stress, and anchorage may be modified as in Sections 402(a), 405(a), and 406(a) for units made by factory methods and under factory control.

(*c*) With reference to precast systems the design of which does not conform to Section 101(b), the sponsors have the right to present for acceptance the data upon which their design is based, in accordance with the provisions of Section 103(a) of ACI 318-56.†

*Adopted as a standard of the American Concrete Institute at its 54th annual convention, Feb. 25, 1958, as reported by Committee 711; ratified by letter ballot June 1, 1958. Title No. 55-4 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Title No. 55-4 supersedes Title No. 50-1 published in Sept. 1953. Separate prints in covers are available at 50 cents each to ACI members ($1.00 to nonmembers). **Discussion** of this report (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†See appendix hereto, p. 90.

CHAPTER 2—MATERIALS

201—Aggregate

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (ASTM C 33-56T) and "Tentative Specifications for Lightweight Aggregate for Structural Concrete" (ASTM C330-53T) provided, however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, watertightness, fire resistance, and wearing qualities may be used under Section 302(a) Method 2, ACI 318-56, when permitted by the authority having jurisdiction.

(*b*) The maximum size of the aggregate shall in no case be larger than three-fourths of the minimum clear distance between reinforcing bars and sides of the forms, nor larger than one-third of the narrowest dimension between sides of the forms in which the unit is cast, except that when concrete is compacted by means of pressure or high-frequency vibration the maximum size of the aggregate shall not be larger than one-half the narrowest dimension between the sides of the forms.

202—Concrete

(*a*) The quality and strength of concrete for floor or roof units made of sand and gravel, crushed stone, air-cooled slag, or other heavy aggregate shall be as provided in Sections 301 and 302 of ACI 318-56.

(*b*) Lightweight concrete for roof or floor units, which is made either by the introduction of air bubbles into the mix (cellular concrete) or by the use of lightweight aggregates, is permitted if the allowable unit stresses used in design satisfy the requirements of Section 401.

203—Reinforcement

(*a*) In the non-prestressed or partially prestressed types of floor or roof units, the reinforcement shall conform to the specifications outlined in ACI 318-56, Section 208. The allowable stresses shall conform to the requirements set forth in Section 306(a) and (b) of ACI 318-56. For fully prestressed units refer to "Tentative Recommendations for Prestressed Concrete," reported by ACI-ASCE Committee 323, ACI JOURNAL, Jan. 1958, *Proc.* V. 54.

CHAPTER 3—TESTS

301—Tests of an individual flexural unit

(*a*) When an individual floor or roof unit is to be tested as a simple span beam, the zero for deflection shall be under the design total dead load to be carried. The maximum 24-hr midspan deflection due to a test load of twice the design live load (with minimum test loads of 80 and 60 psf for floors and roofs respectively) shall not exceed 1/160 of the span, and the residual deflection immediately after removing the test load shall not exceed 1/400 of the span. Such units shall then be tested to complete failure, which shall not occur at less than two times the sum of design dead and live load, nor less

than three times the design live load alone. If no failure fracture occurs the load causing a deflection of 1/60 of the span is to be considered the failure load. Refer also to Section 404(a).

302—Manufacturer's certification

(*a*) A manufacturer whose standard precast floor and roof units have been designed and manufactured in conformity with this ACI standard, and who so represents, shall make available to the prospective customer (or his technical representative) a report which so indicates. He shall also submit a copy of a test report by an approved independent laboratory or consulting engineer covering a Section 301(a) test of a unit from the same manufacturing series as the one under consideration.

303—Load tests of an existing structure

(*a*) When a field test* is to be made, the floor or roof panel shall be tested in accordance with and shall conform to the requirements of ACI 318-56 Sections 202 and 203.

CHAPTER 4—DESIGN

400—Notation

n = ratio of modulus of elasticity of steel to that of concrete
k = ratio of depth of beam neutral axis to the beam depth to centroid of the tensile reinforcement
j = ratio of distance between centroid of compression and centroid of tension to the beam depth d

401—Allowable design stresses in concrete and reinforcement

(*a*) For all concretes the design allowable stresses in the concrete and reinforcement under safe working load shall conform to the requirements of ACI 318-56.

(*b*) For lightweight aggregate concretes and cellular concrete the n values shall be determined as in Section 407(a), and the corresponding values of k and j computed.

(*c*) Fully prestressed units shall be designed according to prestressed beam principles and methods. Refer to "Tentative Recommendations for Prestressed Concrete," reported by ACI-ASCE Committee 323, ACI JOURNAL, Jan. 1958, *Proc.* V. 54.

402—Concrete protection for reinforcement

(*a*) Precast floor and roof units made of high quality factory-controlled concrete and mild steel reinforcement may, when used in locations protected from the weather or moisture, be approved with $\frac{5}{8}$-in. concrete cover for the reinforcement, provided however, that the concrete cover in all cases shall be

*Herein a *field test* is a test made upon a floor or roof area in place in a structure.

at least equal to the diameter of the bars. To insure exact final location of the reinforcement, positive and rigid devices for that purpose shall be used in the manufacturing process. When the precast members are exposed to weather, moisture, or direct fire hazard the protective cover shall be increased to conform with ACI 318-56, Section 507.

403—Structural concrete toppings

(*a*) When field-placed toppings are to be relied upon in computing the strength of a floor section, they must be thoroughly bonded to the surface* of the precast unit. The topping concrete shall develop at least three-fourths of the design strength of that of the precast unit, and have its design governed by the corresponding lower allowable stresses.

(*b*) The maximum aggregate size shall not exceed one-third the thickness of the topping.

(*c*) In no case shall the calculated horizontal shearing stress within the plane of the construction joint exceed 0.02 f_c' of the weaker concrete, due to the design load above it, without the use of shear reinforcement across said plane.

404—Deflections and span-depth ratios

(*a*) The permissible simple span deflections of floor and roof units are set by the test requirements of Section 301(a), but in no case shall the span-depth ratio for floor units exceed 40. For this purpose the thickness of a topping may be included in computing the depth if integral action with the precast portion has previously been achieved through control of strength and placing in conformity with the requirements of Section 403.

405—Flexure bar spacing and bond stress

(*a*) When the reinforcement for precast joists consists of an upper and a lower chord with properly designed web reinforcement welded thereto, and with a vertical bar of the same size as the web reinforcement (but never of less than ⅜ in. diameter) weld-connecting the chords at their ends, the 6-in. anchorage requirement of ACI 318-56, Section 902(c), and the spacing requirement of Section 505(a) may be waived, provided that the members are made under factory controlled conditions in which the concrete is vibrated or forced into place under pressure, and provided that the clear bar spacing is not less than one-half the maximum size of the coarse aggregate, and that the bond stress allowed by ACI 318-56, Section 305,† is not exceeded when computed by the formula:

$$u = \frac{V}{\frac{7}{8} \sum o \, jd}$$

*Satisfactory bonding requires a clean rough surface treated with neat cement grout. The topping must next be immediately placed, thoroughly compacted, and cured.

†See Appendix, p. 93.

406—Anchorage of reinforcement in partially prestressed units

(*a*) The adequacy of the anchorage of the flexure reinforcement of precast floor and roof units having no web end vertical or top reinforcement but which meet the other requirements of Section 405(a) and have flexure reinforcement tensioned in the manufacturing process (or the concrete compressed), shall be demonstrated by suitable flexure tests of the units resulting in failures in flexure or shear rather than of the anchorage.

407—Lightweight concrete *n* values

(*a*) For lightweight aggregate concrete, and for cellular concrete, the value of n to be used in designing shall be determined from rapidly* conducted compression tests of standard 6 x 12-in. cylindrical specimens from the proposed mix. In no case shall the n values used be less than $1\frac{1}{2}$ times the value specified in Section 305 of ACI 318-56 for natural stone concrete of the same compressive strength.

408—Precast joist design

(*a*) Precast joists that will be keyed a minimum of $\frac{1}{2}$ in. into a field-placed floor slab may be erected without falsework or jacking and designed for T-beam action with the slab, provided that in designing the whole dead load is assigned to the precast joist, with only the live load considered to be carried by the combined section of increased depth.

(*b*) When the horizontal shear stress v_h between floor slab and joist (the joists being embedded a minimum of $\frac{1}{2}$ in.) exceeds $0.03\ f_c'$ of the weaker concrete (maximum value 90 psi) due to the superimposed live load, web reinforcement must be used between such elements. See the table of ACI 318-56, Section 305(a).

(*c*) Where the ends of joists do not bear upon walls or girders, as at stair wells, steel joist hangers shall be used to provide end support, or hanger-type bent bars shall be cast into the end of the joist to be supported.

(*d*) The maximum clear distance between the permanent lateral supports of the flanges of a precast joist shall not exceed 32 times the width thereof.

409—Minimum thickness of precast joist slabs

(*a*) The minimum thickness of cast-in-place slabs, for joist heads embedded not less than $\frac{1}{2}$ in. and for joist spacings less than 30 in., is 2 in. For joist spacing of 30 to 36 in. the minimum thickness of slab concrete floors is $2\frac{1}{2}$ in. Greater thickness may be required where unusual loads or spans are encountered. The required thickness of slabs spanning more than 36 in. shall be determined by accepted design methods, but shall not be less than $2\frac{1}{2}$ in. In the case of precast slabs of ribbed or channel section, the thickness requirement applies to the portion thereof containing the tensile reinforcement.

*The specimen shall be preliminarily loaded at least three times to about half its ultimate strength to eliminate initial strains. The stress-strain data shall be taken with the head of the test machine moving continuously at a uniform speed of from 0.025 to 0.05 in. per min.

410—Holes in webs of joists

(*a*) Holes in the webs of joists shall be formed at the plant, or carefully cut (not punched) after approval by and under the supervision of the architect or engineer. For additional requirements see Chapter 8.

411—Hollow-core floor and roof unit design

(*a*) Hollow-core precast units shall be connected laterally by continuous grout keys and/or close fitting mechanical keys capable of forcing adjacent units to act together.

(*b*) Abutting precast hollow-core type units, which are to be completely covered by a field-placed concrete topping slab of controlled strength and placing, Section 403(a), may be installed without falsework or jacking (as in the case of bare units) *provided* that, in designing, the total dead load is assigned to the bare unit, with only the live load considered to be carried by the final topped section of increased depth.

412—Extra or concentrated loads

(*a*) Where a joist system supports partition walls parallel to the joists, or where heavy concentrated loads are to be expected, joists may be placed side by side if adequate provision is made to insure approximately equal loading of all joists.

(*b*) Where a hollow-core (closed section) system supports partition walls parallel to the unit, or where heavy concentrated loads are to be expected, such loads may be considered to be uniformly distributed over not more than three identical units on each side thereof but never over a greater total width than 0.4 of the clear span distance.

413—Assembled block floor and roof units—Design and tests

(*a*) These assemblies are essentially partially prestressed flexural members in which uniform contact between individual block is achieved by face grinding or other means. The partial prestress is obtained by the use of steel meeting the requirements of ACI 318-56, Section 306(a).

(*b*) When it appears that the shear strength of an assembled unit may be less than required, the ultimate strength thereof shall be demonstrated by simple span tests of random sample assemblies to failure over the minimum and maximum spans involved.

(*c*) The strength of the end anchorages of the bars shall meet the requirements of Section 406(a).

(*d*) If topping is used, and relied upon for flexural strength, Section 411(b) for hollow-core units applies. Refer also to Section 301(a) which applies to all precast floor and roof units.

(*e*) Chapters 7 and 8, regarding installation and openings, apply to these units, as to the others.

CHAPTER 5—CURING AND HANDLING

501—Curing

(*a*) Immediately after molding, the precast units shall be cured by one of the methods of Section 501(b), or by keeping them moist at a temperature of 70 F or more for at least 7 days if made of normal portland cement, and for at least 3 days if made of high-early-strength portland cement.

(*b*) Curing by high pressure steam, steam vapor, or other accepted processes may be employed to accelerate the hardening of the concrete and to reduce the time of curing provided, however, that the specified minimum compressive strength is attained, and that its 28-day strength exceeds its earlier strengths.

(*c*) Control cylinders shall be cured under the same conditions as the precast units themselves.

502—Handling

(*a*) Units shall be stored on suitably prepared supports, free from warp. They shall not be delivered until of sufficient strength to be safely transported. They shall be carefully placed in final position without overstressing or damage.

CHAPTER 6—MANUFACTURE

601—Workmanship

(*a*) The finished product shall be free of honeycomb or rock pockets. The mix, gradation of the aggregate, and workability shall be such as to insure complete filling of the form and continuous intimate bond between the concrete and all steel.

602—Identification and marking

(*a*) All floor and roof units shall bear a permanent identifying symbol as well as a mark indicating the top of the unit. The identifying symbol shall be the same one used for the unit in the manufacturer's literature and shall be shown in a table on the setting plan together with the length, type, and size of unit and the amount, size, and arrangement of all reinforcement. The tabulated information shall be complete enough to permit the calculation of the load capacity of the unit.

CHAPTER 7—INSTALLATION

701—Installation and construction details

(*a*) The engineer or architect shall provide a stress sheet which shows the manufacturer the maximum bending moments and shears at the critical sections, except in cases in which the particular span-length unit and its loads fall within the limitations of the load table which has already been approved.

(*b*) A setting plan shall be prepared for every job by the manufacturer for the approval of the architect or engineer. The setting plan shall show the grade of reinforcing steel, the type of aggregate, and the strength of the con-

crete to be used. If the units are designed as prestressed members, the ultimate tensile strength of the reinforcement, the proportionality limit thereof, and the initial prestress to be used shall also be stated. The plan shall include the locations of all openings, and the headers to be provided.

CHAPTER 8—HOLES AND OPENINGS

801—Requirements at structural discontinuities

(*a*) No holes in precast units shall be made on a job site except after the approval by and under the supervision of the architect or engineer. All holes shall be formed or carefully cut, not punched.

(*b*) Holes in the bottom or top faces of hollow units, for conduit hangers or pipes, may be field cut through the hollow portion of the unit provided that not less than 1 in. clear distance from the longitudinal reinforcement is maintained.

(*c*) Transverse channeling in the top or bottom faces of units, except over a support, is prohibited. Channels over supports must be so located that the shear strength of the unit is not decreased below that required by this standard.

(*d*) Where a single hollow-core (closed section) unit meeting the requirements of Section 411(a) is cut, the uniformly distributed load originally considered to be carried by the unit may be considered to be laterally distributed as specified in Section 412(b). One or more cut units carrying partitions or other concentrated loads shall be supported by specially designed reinforced headers or curbs. Wherever more than a single unit is cut a header or curb shall be used and the requirements of Section 412(b) shall be satisfied.

APPENDIX

Excerpts from "Building Code Requirements for Reinforced Concrete" (ACI 318-56)

103—Special systems of reinforced concrete

(*a*) The sponsors of any system of design or construction of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by this code shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Building Official. This Board shall be composed of competent engineers, architects, and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Building Official shall be of the same force and effect as the provisions of this code.

200—Notation

D = deflection, produced by a test load, of a member relative to the ends of the span
L = span of member under load test (the shorter span of flat slabs and of floors supported on four sides)
t = total thickness or depth of a member under load test

202—Load tests of existing structures

(*a*) A load test of an existing structure to determine its adequacy (stiffness and strength) for the intended use shall not be made until the portion subjected to the load is at least 56 days old, unless the owner of the structure agrees to the test being made at an earlier age.

(*b*) When a load test is required and the whole structure is not to be tested, the portion of the structure thought to provide the least margin of safety shall be selected for loading. Prior to the application of the test load, a load which simulates the effect of that portion of the design dead load which is not already present shall be applied and shall remain in place until after a decision has been made regarding the acceptability of the structure. The test load shall not be applied until the structural members to be tested have borne the full design dead load for at least 48 hr.

(*c*) Immediately prior to the application of the test load, the necessary initial readings shall be made for the measurements of deflections (and strains, if these are to be determined) caused by the application of the test load. The members selected for loading shall be subjected to a superimposed test load of two times the design live load, but not less than 80 psf for floor construction nor less than 60 psf for roof construction. The superimposed load shall be applied without shock to the structure and in a manner to avoid arching of the loading materials. Unless otherwise directed by the Building Official, the load shall be distributed to simulate the distribution of the load assumed in the design.

203—Loading and criteria of acceptability

The test load shall be left in position for 24 hr when readings of the deflections shall again be made. The test load shall be removed and additional readings of deflections shall then be made 24 hr after the removal of the test load. The following criteria shall be used in determining conformity with the load test requirements.

(*a*) If the structure shows evident failure, the changes or modifications needed to make the structure adequate for the rated capacity shall be made; or a lower rating may be established.

(*b*) Floor and roof construction shall be considered to conform to the load test requirements if there is no evidence of failure and the maximum deflection does not exceed:

$$D = \frac{L^2}{12{,}000\ t} \qquad (1)$$

in which all terms are in the same units. Constructions with greater deflections shall meet the requirements of subsections (c), (d), and (e).

(*c*) The maximum deflection of a floor or roof construction shall not exceed the limit in Table 203(c) considered by the Building Official to be appropriate for the construction.

TABLE 203(c)—MAXIMUM ALLOWABLE DEFLECTION

Construction	Deflection
1. Cantilever beams and slabs	$L^2/1800\ t$
2. Simple beams and slabs	$L^2/4000\ t$
3. Beams continuous at one support and slabs continuous at one support for the direction of the principal reinforcement	$L^2/9000\ t$
4. Flat slabs (L = the longer span)	$L^2/10{,}000\ t$
5. Beams and slabs continuous at the supports for the direction of the principal reinforcement	$L^2/10{,}000\ t$

(*d*) The maximum deflection shall not exceed $L/180$ for a floor construction intended to support or to be attached to partitions or other construction likely to be damaged by large deflections of the floor.

(*e*) Within 24 hr after the removal of the test load the recovery of deflection caused by the application of the test load shall be at least 75 percent of the maximum deflection if this exceeds

$L^2/12{,}000\ t$. However, constructions failing to show 75 percent recovery of the deflection may be retested. The second test loading shall not be made until at least 72 hr after the removal of the test load for the first test. The maximum deflection in the retest shall conform to the requirements of Sections 203(c) and (d) and the recovery of deflection shall be at least 75 percent.

301—Concrete quality

(*a*) For the design of reinforced concrete structures, the value of f_c used for determining the allowable stresses as stipulated in Section 305 shall be based on the specified minimum 28-day compressive strength of the concrete, or on the specified minimum compressive strength at the earlier age at which the concrete may be expected to receive its full load. All plans, submitted for approval or used on the job, shall clearly show the assumed strength of concrete at a specified age for which all parts of the structure were designed.

(*b*) Concrete without air entrainment which will be exposed to the action of freezing weather shall have a water content not exceeding 6 gal. per sack of cement.*

302—Methods for determining strength of concrete

(*a*) The determination of the proportions of cement, aggregate, and water to attain the required strengths shall be made by one of the following methods:

Method 1—Without preliminary tests

Where preliminary test data on the materials to be used in the concrete are not available, the water-cement ratio shall not exceed the values shown in Table 302(a).† When strengths in excess of 4000 psi are required or when lightweight aggregates or admixtures (other than those exclusively for the purpose of entraining air) are used, the required water-cement ratio shall be determined in accordance with Method 2.

TABLE 302(a)—PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE

Specified minimum compressive strength at 28 days, psi	Maximum permissible water-cement ratio, U.S. gal. per 94-lb sack of cement*	
	Non-air-entrained concrete	Air-entrained concrete
2000	8	7¼
2500	7¼	6¼
3000	6½	5¼
3500	5¾	4½
4000	5	4

*Including free surface moisture on aggregates.

Method 2—With preliminary tests

Water-cement ratios or strengths greater than shown in Table 302(a) may be used provided that the relationship between strength and water-cement ratio for the materials to be used has been previously established. Where previous data are not available, concrete of proportions and consistency suitable for the work shall be made using at least three different water-cement ratios which will produce a range in strengths encompassing those required for the work. These tests shall be made in accordance with the procedure given in the appendix to "Recommended Practice for Selecting Proportions for Concrete" (ACI 613). For each water-cement ratio, at least three specimens shall be made and cured in accordance with "Method of Making and

*Detailed recommendations for quality of concrete and requirements for air content for various exposures are given in "Recommended Practice for Selecting Proportions for Concrete" (ACI 613).

†The tabulated water-cement ratios are more conservative than those given in ACI 613 and will generally produce appreciably higher strengths than indicated.

Curing Concrete Compression and Flexure Test Specimens in the Laboratory" (ASTM C 192) and tested for strength in accordance with "Method of Test for Compressive Strength of Molded Concrete Cylinders" (ASTM C 39).

The strength tests shall be made at 28 days or the earlier age at which the concrete is to receive its full working load, as indicated on the plans. A curve shall be established showing the relationship between water-cement ratio and compressive strength. The maximum permissible water-cement ratio for the concrete to be used in the structure shall be that shown by the curve to produce a strength 15 percent greater than called for on the plans, except when a lower value of the water content is required by Section 301(b). Where different materials are to be used for different portions of the work, each combination shall be evaluated separately.

305—Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used when designs are made in accordance with Section 601(a) shall not exceed the values of Table 305(a) where f_c' equals the minimum specified compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable unit stresses						
		For any strength of concrete in accordance with Section 302 $n = \frac{30{,}000}{f_c'}$	Maximum value, psi	For strength of concrete shown below				
				f_c' = 2000 psi n = 15	f_c' = 2500 psi n = 12	f_c' = 3000 psi n = 10	f_c' = 3750 psi n = 8	f_c' = 5000 psi n = 6
Flexure: f_c								
Extreme fiber stress in compression	f_c	$0.45f_c'$		900	1125	1350	1688	2250
Extreme fiber stress in tension in plain concrete footings	f_c	$0.03f_c'$		60	75	90	113	150
Shear: v (as a measure of diagonal tension)								
Beams with no web reinforcement	v_c	$0.03f_c'$	90	60	75	90	90	90
Beams with longitudinal bars and with either stirrups or properly located bent bars	v	$0.08f_c'$	240	160	200	240	240	240
Beams with longitudinal bars and a combination of stirrups and bent bars (the latter bent up suitably to carry at least $0.04f_c'$)	v	$0.12f_c'$	360	240	300	360	360	360
Footings*	v_c	$0.03f_c'$	75	60	75	75	75	75
(For flat slabs, see Chapter 10)								
Bond: u								
Deformed bars (as defined in Section 104)								
Top bars†	u	$0.07f_c'$	245	140	175	210	245	245
In two-way footings (except top bars)	u	$0.08f_c'$	280	160	200	240	280	280
All others	u	$0.10f_c'$	350	200	250	300	350	350
Plain bars (as defined in Section 104) (must be hooked)								
Top bars	u	$0.03f_c'$	105	60	75	90	105	105
In two-way footings (except top bars)	u	$0.036f_c'$	126	72	90	108	126	126
All others	u	$0.045f_c'$	158	90	113	135	158	158
Bearing: f_c								
On full area	f_c	$0.25f_c'$		500	625	750	938	1250
On one-third area or less‡	f_c	$0.375f_c'$		750	938	1125	1405	1875

*See Sections 905 and 809.

†Top bars, in reference to bond, are horizontal bars so placed that more than 12 in. of concrete is cast in the member below the bar.

‡This increase shall be permitted only when the least distance between the edges of the loaded and unloaded areas is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third but less than the full area shall be interpolated between the values given:

306—Allowable unit stresses in reinforcement

Unless otherwise provided in this code, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = tensile unit stress in longitudinal reinforcement)
and (f_v = tensile unit stress in web reinforcement)

20,000 psi for rail-steel concrete reinforcing bars, billet-steel concrete reinforcing bars of intermediate and hard grades, axle-steel concrete reinforcing bars of intermediate and hard grades, and cold-drawn steel wire for concrete reinforcement.

18,000 psi for billet-steel concrete reinforcing bars of structural grade, and axle-steel concrete reinforcing bars of structural grade.

(*b*) *Tension in one-way slabs of not more than 12-ft span*

(f_s = tensile unit stress in main reinforcement)

For the main reinforcement, ⅜ in. or less in diameter, in one-way slabs, 50 percent of the minimum yield point specified in the specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 psi

(*c*) *Compression, vertical column reinforcement*

(f_s = nominal allowable stress in vertical column reinforcement)

Forty percent of the minimum yield point specified in the specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 psi.

(f_r = allowable unit stress in the metal core of composite and combination columns)

Structural steel sections	16,000 psi
Cast iron sections	10,000 psi
Steel pipe	See limitations of Section 1106(b)

(*d*) *Compression, flexural members*

For compression reinforcement in flexural members see Section 706(b).

507—Concrete protection for reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than 3 in. of concrete between it and the ground contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than 2 in. of concrete for bars larger than #5 and 1½ in. for #5 bars or smaller.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than ¾ in. for slabs and walls; and not less than 1½ in. for beams, girders, and columns. In concrete joist floors in which the clear distance between joists is not more than 30 in., the protection of reinforcement shall be at least ¾ in.

(*c*) If the general code of which this code forms a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of bars.

(*e*) Exposed reinforcing bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

Discussion of this report should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

Title No. 55-5

Origin, Evolution, and Effects of the Air Void System in Concrete. Part 1—Entrained Air in Unhardened Concrete*

By RICHARD C. MIELENZ,† VLADIMIR E. WOLKODOFF,‡ JAMES E. BACKSTROM,§ and HARRY L. FLACK**

Entrained air benefits concrete mainly in two general ways: (1) improvement of the workability and decrease in segregation of freshly mixed concrete, and (2) improved resistance of the concrete to freezing and thawing. Studies of these phenomena in the Engineering Laboratories, Bureau of Reclamation, United States Department of the Interior, are summarized in four reports, the first of which is here presented. The findings justify additional investigation along several interrelated lines, such as systematic study of the bubble system in freshly mixed concrete and its function during working and placing; changes in the bubble system after placing and before hardening of the concrete; measurement of the geometry of the void system in hardened concrete; development of new analytical procedures and tests for evaluating the role of entrained air voids in exposed concrete; specifications for air-entraining admixtures and cement; and, in general, procedures by which satisfactory and uniform air-entrained concrete can be produced consistently.

The action of air bubbles in freshly mixed concrete depends primarily upon the number, size distribution, and spacing of the bubbles maintained during mixing, placing, and finishing. T[illegible] characteristics of the void system are determined by the ability of the concrete to develop within itself during mixing an abundance of air bubbles which resist dissolution, coalescence, and loss by escape from the concrete. Of fundamental importance are the composition and proportion of air-entraining agent employed, the water-cement ratio, and the method and intensity of compaction. These factors are examined and their significance is discussed in Part 1 of this series.

As has been shown previously by others,[1] microscopical examination of prepared surfaces permits determination of the approximate size and spacing of the voids in hardened concrete. Parts 2 and 3 of this series will present data obtained from studies of the void system in hardened concrete in relation to important variables in mixing and placing operations. Part 4 summarizes evaluation of the void system in concrete taken from field construction. These data permit a generalization concerning the validity of the procedures stipulated in "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)"[2] so far as they relate to air-entrained concrete.

Although the concepts and conclusions presented here are justified by available experimental data and theory, considerable additional work is necessary to provide more quantitative information on the various factors and processes involved and to investigate the various significant implications of the concepts, especially as they relate to procedures by which the quality of air-entrained concrete can be improved and made more uniform.

*Received by the Institute Aug. 8, 1957. Title No. 55-5 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Member American Concrete Institute, Director of Research, Master Builders Co., Cleveland, Ohio; formerly Head, Petrographic Laboratory, Bureau of Reclamation, Denver, Colo.

‡Member American Concrete Institute, Senior Engineer, The Carborundum Co., Niagara Falls, N. Y., formerly Petrographer, Petrographic Laboratory, Bureau of Reclamation, Denver, Colo.

§Member American Concrete Institute, Head, Materials, Mixes, and Durability Section, Bureau of Reclamation, Denver, Colo.

**Engineer, Bureau of Reclamation, Denver, Colo.

SYNOPSIS

Air in concrete originates in four general ways, producing "entrapped" and "entrained" voids which are differentiated by their spatial and pressure relationships to the surrounding water and solids. The action of air-entraining agents in modifying the void system in concrete is discussed. Once formed, the air void system deteriorates in characteristic ways, mainly by interchange of air between small bubbles and large, and by loss of a portion of the air during compaction of the concrete. Several factors determine the rate and extent of deterioration effected before hardening of the concrete are described.

The action of the air void system in improving workability and decreasing segregation and bleeding is discussed.

INTRODUCTION

The extent to which air-entrained concrete differs from non-air-entrained concrete depends primarily upon the size and spacing of air bubbles that develop in the concrete during mixing, and the rate at which the bubbles change in number and size during handling and placing and prior to hardening of the concrete. Since the void system in hardened concrete is only a cast of the system of bubbles present at the time of hardening, it is instructive to examine the processes that control the production of bubbles and their subsequent history in the concrete prior to hardening. When they are complete, such studies should define quantitatively the necessary attributes of air-entraining admixtures and air-entraining cements, and so make possible more scientific production and evaluation of air-entraining agents and air-entrained concrete.

The answers to many of the questions to be resolved lie in fields of chemistry of surface-active agents; the physical chemistry of foams, solutions, and slurries; and details of the processes of mixing, handling, and placing of concrete. These subjects are complex and not completely understood even in comparatively simple systems. Hence, the authors wish to stress that they are not attempting a thorough dissertation on the formation and development of the void system in concrete. Rather, they believe that thinking about these matters has led to concepts which are worthy of reporting so that they may be available for review and discussion and so that, especially, they may stimulate similar work by others.

ORIGIN OF AIR IN CONCRETE

Air present in voids in the cement paste of unhardened concrete is derived from several sources: (1) air originally present in intergranular spaces in the cement and aggregate; (2) air originally present within the particles of cement and aggregate but expelled from the particles before hardening of the concrete by inward movement of water under hydraulic and capillary potential; (3) air originally dissolved in the mixing water; and (4) air which is in-folded and mechanically enveloped within the concrete during mixing and placing. These

are the only sources of air bubbles in concrete, whether or not an air-entraining agent is used.

A large proportion of the air bubbles generated by the mixing and placing operations remains within the concrete because the bubbles are enmeshed within the packed aggregate and because they may adhere to particles of cement or, less commonly, to particles of aggregate. For a well proportioned mixture, the aggregate in the placed air-entrained concrete constitutes a virtually continuous granular structure, within which the cement paste and air voids are intercalated. Air voids escape from this granular structure only by moving between intergranular spaces under the force of buoyancy as the aggregate particles are shifted with respect to one another during placing and compaction of the concrete.

AIR VOIDS IN CONCRETE

Air voids in concrete range in their relationship to the granular structure of concrete between two extremes: on the one hand, bodies of air entirely enclosed among particles of aggregate may be under pressure approaching that imposed only by the overlying water; at the other extreme, bubbles enclosed by a paste composed of aggregate fines and cement will be under pressure approaching that imposed by the entirety of the overlying concrete. Actually, air voids in unhardened concrete or mortar typically will be under pressure intermediate between these extremes.

The former type of air void commonly is said to be "natural" or "entrapped." The term "natural air voids" is misleading because there is no evidence that any particular types of voids are inherent constituents of a combination of aggregate, cement, and water, independent of the presence or absence of an air-entraining agent or degree of compaction. The term "entrapped air" to designate these voids is also inappropriate because all air voids in plastic concrete are literally entrapped by any of several mechanisms. However, this latter term will be employed for simplicity because of its wide usage. These voids characteristically are 1 mm or more in diameter and irregular in shape because the periphery of the void follows the contour of the surrounding aggregate particles. Voids of this type are most abundant in non-air-entrained lean concrete containing sand deficient in fines. Air in such voids is under pressure imposed by the capillary forces and hydrostatic pressure of the water in the mixture because the weight of the solid matter in the overlying concrete is borne by the granular framework developed by the aggregate. These voids are ineffective in improving workability of concrete because they do not decrease, and they may increase, the dilatancy of the mass necessary to manipulation.

"Entrained" air voids—the second type—are retained in the concrete as a result of entrapment among the aggregate, adherence by surface-chemical forces to particles of cement and aggregate, and the viscosity of the cement paste. These voids typically are between 10 and 1000 microns in diameter,

and spherical or nearly so because of the hydrostatic pressure to which they are subjected by the surrounding paste of water, cement, and aggregate fines. These voids are best developed in air-entrained rich concrete containing well graded sand. Entrained air bubbles effectively improve workability in the unhardened concrete because they increase the spacing of solids in the mass and thus decrease dilatancy and, by bearing short time loads, they facilitate movement of aggregate particles past one another.

The air content of a concrete mixture may be altered in any of several ways, including change of the grading of the aggregate, change of cement and water content, variation in the kind and amount of air-entraining agent, change of temperature, and variation in degree of compaction of the concrete. However, a given change in air content resulting from such alterations in mixture proportions and placing conditions will not effect equivalent changes in the properties of the fresh or hardened concrete. This is true primarily because these changes affect the frequency and relative volume of the two types of voids to a varying degree. On one hand, reduction of the abundance and size of the "entrapped" voids is primarily a matter of compaction of the concrete, or of so modifying the proportions of a concrete mixture as to cause space otherwise filled by air to be occupied by cement, aggregate, and water. Such a change will increase strength of the hardened concrete, but will not materially modify resistance to freezing and thawing. Conversely, reduction in the abundance of entrained air bubbles removes a unique structural element from the concrete and effects substantial changes in the properties of both the fresh and hardened concrete.

As had been shown previously by many investigators,[3,4] air content of concrete is increased by increase in the proportion of sand of intermediate size, particularly in the ranges passing the No. 30 and retained on the No. 50 or the No. 100* sieves, and is decreased by increase in coarser or finer sizes of aggregate or of cement. This size fraction is critically important because the openings between grains in this size range are small enough to impede movement of portland cement, yet the grains are large enough to retain enmeshed among their granular framework bubbles big enough to withstand rapid dissolution in the mixing water in the presence of a satisfactory air-entraining agent. In a granular mass composed of spherical particles of uniform size in the most compact arrangement, the diameter of the largest sphere that can be contained in the interstitial space is 0.22 times the particle diameter. Lack of uniform size and departure from sphericity will decrease the size of the enclosed space if the most compact arrangement is achieved.

The maximum size of the space subtended by particles passing the No. 30 and retained on the No. 100 sieve varies from about 33 to 130 microns. As will be shown in subsequent sections, 90 to 99 percent of the air voids in air-entrained concrete are less than 100 microns in diameter and so can be contained among sand particles in this size range. Moreover, it will be shown that

*The precise range found to be most effective probably will vary with the characteristics of each fine aggregate, particularly with respect to grading within this range, particle shape, surface texture, and mineralogic composition.

bubbles larger than about 60 microns are far more susceptible to escape from the concrete during vibration than are smaller ones, there being a rather sharp change in the rate of loss at about this size. An increase in the proportion of sand finer than that which is optimum for entraining air will decrease the available volume among the particles in this fraction and will consequently subject the displaced bubbles to greatly increased pressure, thus facilitating their escape from the concrete and increasing the solubility of the air and hastening their dissolution. An increase in coarser aggregate decreases the available interstitial space of optimum dimensions because a portion of the finer sand is replaced by aggregate particles.

Hence, it appears that clusters of grains in approximately the No. 30-50 or No. 30-100 size range are an important reservoir for those bubbles which persist until hardening of the concrete.

ACTION OF AIR-ENTRAINING AGENTS IN SOLUTIONS AND PORTLAND CEMENT SLURRIES

Adsorption

In the concentrations ordinarily used in concrete, air-entraining agents are positively adsorbed at air-water interfaces and reduce the surface tension, possibly by 20 dynes per cm or more. A substance in solution is said to be positively adsorbed if it is more concentrated at a surface or interface than it is in the solution as a whole. The "film" of air-entraining agent so produced can effect important changes in the air bubbles in a liquid or slurry because coalescence of bubbles and rate of dissolution of the air in the bubbles are reduced to an extent depending in detail upon the concentration and properties of the film-forming substance.

Precipitation of insoluble calcium salts

In the presence of portland cement or other substances on which or from which alkali earth or metallic ions are available, some types of air-entraining agents are affected in critical ways. If the calcium salt of the surface-active constituent of the air-entraining agent is but slightly soluble in water, the concentration of this compound at the air-water interfaces may be sufficiently high to produce a precipitated solid or gelatinous film which will enclose each air bubble. Such a film will be characterized by finite thickness, strength, elasticity, permeability, and other properties which, together, will control the subsequent response of the air bubble to mechanical or physical-chemical changes in the system. Under ideal conditions such films can be seen microscopically and photographed (Fig. 1).

A preliminary evaluation of the tendency of an air-entraining agent to produce a relatively insoluble precipitate when combined with water and portland cement can be obtained by addition of clear lime water to an aqueous solution of the agent. Agents in which the surface-active constituent is a sodium soap of wood resin, such as neutralized Vinsol resin or sodium abietate;

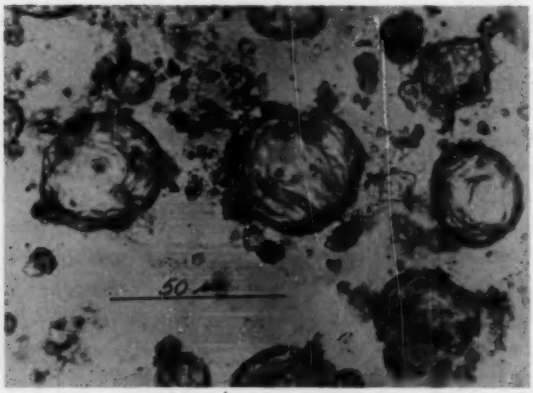


Fig. 1—Photomicrograph of bubbles of air in a dilute slurry of portland cement and a solution of Agent J (Table 1). Solution added repeatedly to mount on glass slide and allowed to evaporate without drying. The bubbles are 3½ hr old

sodium soaps of lignin derivatives, rosin, or fatty acid; or triethanolamine salts of sulfonic acid, produce flocculent white to brown precipitates when saturated lime water is added to an aqueous solution of the air-entraining agent at or above the concentration obtained in the mixing water in the production of air-entrained concrete. This may be demonstrated readily in a small vial or test tube. Of course, such a test is not diagnostic of the quality of an air-entraining agent.

Conversely, many calcium salts of sulfonic acids are soluble in water, and so produce no precipitate with addition of lime water to an aqueous solution. Hence, various air-entraining agents in which the surface-active constitutent is a sulfonate would not be expected to produce a precipitated membrane around the air bubbles in portland cement paste, even though these compounds are positively adsorbed and greatly reduce surface tension of the solution.

Adhesion of bubbles to particles of cement and aggregate

The calcium salt of low solubility precipitated at air-water interfaces in solutions of the air-entraining agents containing sodium salts of abietic acid, lignin, fatty acids, etc., by calcium in solution also will be precipitated on the surface of cement particles, possibly as a monomolecular film. Similar films also may form by adsorption on the surfaces of such particles of organic ions or molecules not producing a precipitate in lime water. Such films will reduce the hydrophilic quality of the surface and may render it hydrophobic. In the latter event, air bubbles will tend to cling to the cement particles with a force determined by the interfacial tension and the contact angle. The greater the contact angle measured in the liquid, the greater the adhesive force.

It would be desirable to develop a definitive procedure to measure the degree of adhesion established between portland cement and bubbles produced by an air-entraining agent in water solutions. Phenomena affording a basis for such evaluation include: (1) measurement of contact angle of the air-water interface against plane surfaces on either portland cement clinker or neat portland cement paste (or analogous calcic compounds or minerals); (2)

evaluation of the ability of froths produced in aqueous solutions to float portland cement, calcic clinker compounds, or calcic minerals; or (3) evaluation of the quantity of portland cement, calcic clinker compounds, or calcic minerals which cling to air bubbles produced in water solutions of air-entraining agents. The authors have investigated each of these possibilities, but no reliable quantitative method has been developed yet. Nevertheless, the observations demonstrate that definite adhesion is established between portland cement or analogous compounds and minerals, and air bubbles produced in solutions of many satisfactory air-entraining agents. Unfortunately, the mechanism underlying adhesion of a bubble to a solid surface is completely unknown.[5]

Measurement of contact angle of the air-water interface against polished surfaces of portland cement clinker or analogous surfaces is an especially promising procedure. Contact angles ranging from virtually nil (little or no adhesion of bubbles to the surface) to as high as 81 deg 30 min have been measured (Fig. 2). Unfortunately, the contact angle produced in a given test is controlled not only by the kind and concentration of the agent and the composition, physical characteristics, and freedom from contamination of the solid, but also by the details of technique and the characteristics of the individual bubble, primarily its size and whether it is shrinking or growing during the experiment. It is hoped that these problems can be resolved by additional investigations.

Another significant area of study concerning bubble-solid adhesion and film formation by surface-active constituents of air-entraining agents involves aggregates. With differing chemical and mineralogic compositions, surface texture, and degree of alteration, aggregates will vary in their effect upon solutions of air-entraining agents. It is most desirable for uniform performance that air-entraining agents be influenced only to minor degree, if at all, by the character of the aggregate. Aggregates in which alkali earth and metallic

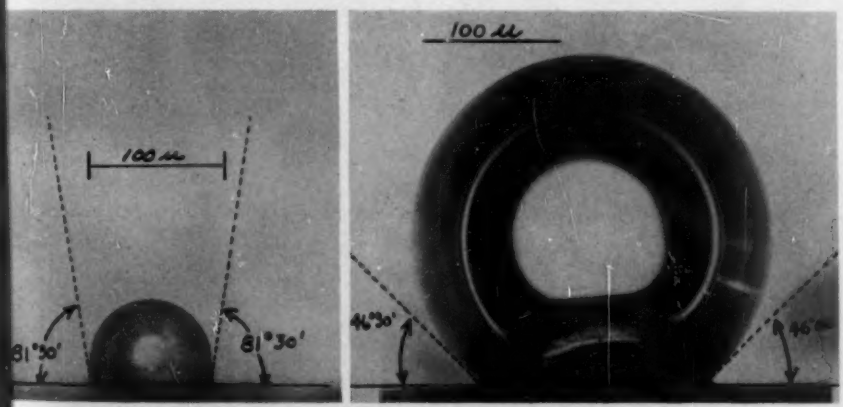


Fig. 2—Contact angle established between air-water interface and a ground surface of portland cement clinker. Agent C at left, Agent J at right. Three drops of agent per 30 ml distilled water

ions are available, such as limestone, dolomite, blast furnace slags, and glassy basalts, are expected to be the most significant in this respect. Additional research must be done to evaluate the magnitude of this effect before engineering decisions are made with respect to it.

It is anticipated that bubble-cement or bubble-aggregate adhesion will be most significant for certain ranges of particle size. Studies of ore flotation indicate that particles in sizes between about 10 and 50 microns are most susceptible to bubble adhesion.[5] For cement, coarse particles probably will be most significant in this respect because they are in the range found best for ore flotation and because the finer cement particles will lose their particulate character by extensive hydration before or shortly after the concrete is placed.

CHARACTERISTICS OF AIR BUBBLES IN SOLUTIONS AND SLURRIES

Nomenclature

a = area of the surface through which air is diffusing from bubbles into the water phase or from the water phase into bubbles, whichever is the smaller

A = volume of voids, fraction of the volume of concrete

α = specific surface of voids

B = mass of air transmitted from small bubbles to larger bubbles by diffusion through the water phase in plastic concrete

d = diameter of air bubble

D = a factor expressing the rate at which air is transmitted through the adsorbed film at the air-water interface and diffuses through the water phase in plastic concrete

γ = surface tension of the water phase in plastic concrete

K = constant from Henry's law

$\bar{l}$ = average chord length, i.e., the average distance across intersected bubbles along a line of traverse

$\bar{L}$ = computed spacing factor

M = true number of bubbles per unit volume of concrete

n = number of voids intersected per unit length of traverse

p = paste content, fraction of the volume of concrete, sum of volumes of water and cement

P = excess pressure resulting from surface tension at the air-water interface in plastic concrete

$(r)_i$ = the i th moment of the true bubble radii in a system containing bubbles of various sizes

r = radius of air bubble

t = time

T = average distance between surfaces of bubbles losing and those gaining air by diffusion in plastic concrete

X = mole fraction of gas dissolved in water

Excess pressure in air bubbles in unhardened concrete

Air enclosed in bubbles in unhardened cement paste in concrete is under greater than atmospheric pressure because of: (1) the hydrostatic pressure of the overlying concrete, or a portion thereof,* and any other load imposed on the concrete, and (2) the curvature of the air-water interface. The surface phenomena are complicated by precipitated films at the interface, but basic relationships are thought to be similar. Such a conclusion is supported by available experimental observations summarized below. Hydrostatic pressure

*A portion or all of the weight of the solids above any particular level in plastic concrete may be borne by the granular framework of aggregate, so that air voids may be subjected only to capillary forces and hydrostatic pressure of the water in some situations.

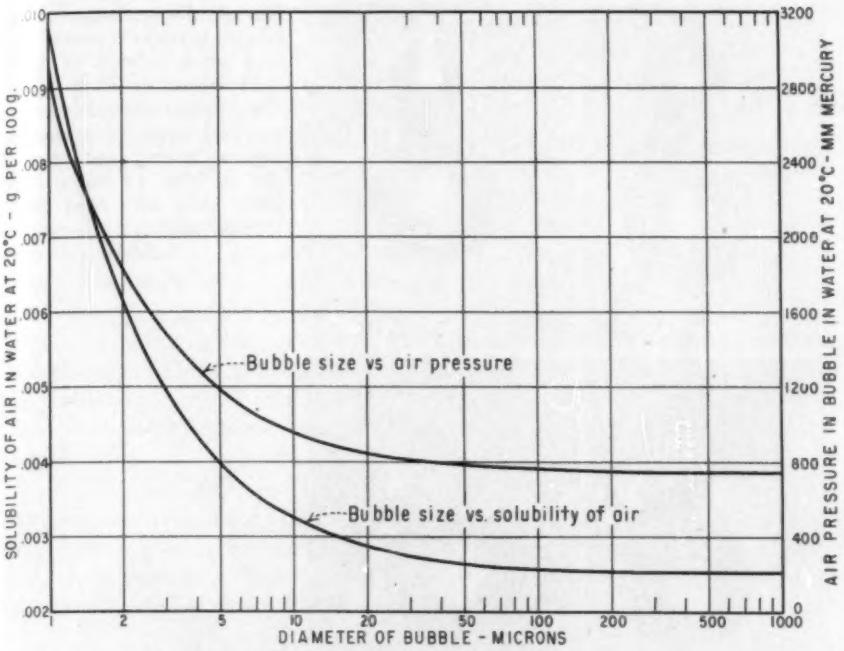


Fig. 3—Relationship of solubility of air and internal pressure to the size of an air bubble in water at 20 C

resulting from weight of the overlying concrete may range from virtually nil to several pounds per square inch, the increase being at the rate of about 1 psi per ft of depth. The pressure resulting from surface tension at the air-water interface is controlled by the relationship:

$$P = \frac{2\gamma}{r} = \frac{4\gamma}{d}$$

Results of calculations for an air-water system at 20 C are summarized in Fig. 3, assuming no superimposed load other than atmospheric pressure. The pressure within an air bubble in water increases rapidly as the diameter of the bubble decreases in the range less than 100 microns. The pressure is 1.70 atmospheres when the diameter is 4 microns and 3.85 when the diameter is 1 micron.

Solubility of air bubbles in water

From the relationship $X = P/K$, where X = mole fraction of gas dissolved in water because of the excess pressure and K = the constant from Henry's law,* it can be shown that the solubility of air in water at 20 C increases from about 250×10^{-5} g per 100 g of water at atmospheric pressure to about

*$K = 5.75 \times 10^7$ for nitrogen and 2.95×10^7 for oxygen at 20 C.[7]

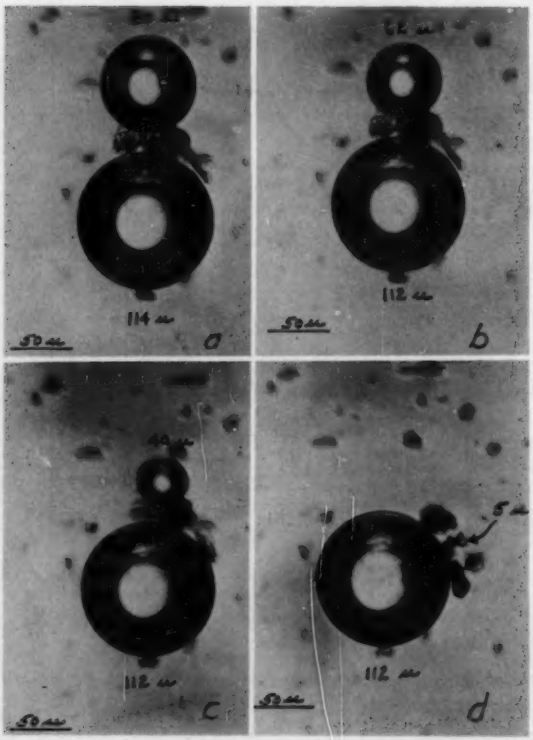


Fig. 4—Photomicrograph of two air bubbles in a saturated lime water solution of Agent C (3 drops per 30 ml of distilled water) with sand passing No. 200 sieve: (a) at completion of mixing, (b) after 9 min, (c) after 14 min, and (d) after 16½ min. Note adhesion of sand particles to the bubbles

430 × 10^{-5} g per 100 g of water at 1.70 atmospheres and to 990 × 10^{-5} g per 100 g of water at 3.85 atmospheres (see Fig. 3). Hence, the tendency for air in bubbles in water to dissolve increases rapidly as the size of the bubbles decreases in the range below 100 microns and the increase is extremely rapid in the range below 10 microns. Consequently, while concrete is unhardened, the air in very small bubbles is being dissolved far more rapidly than is the air in large bubbles, and it is improbable that a bubble originally less than 10 microns will be preserved. Bubbles originally larger than 10 microns will produce voids less than 10 microns in diameter in the hardened concrete because the cement paste hardens after the size of some bubbles has been reduced to that range by dissolution of the air.

Interchange of air in bubbles in solutions, slurries, and concrete

As the air in the small bubbles dissolves, the water soon becomes saturated with dissolved air with respect to bubbles of average radius and the atmosphere above the concrete. From this time until the cement paste becomes sufficiently rigid to produce a relatively stable structure, air in bubbles smaller than the average dissolves and bubbles larger than the average increase in size because of release of the air from the water. This process may be ob-

served easily under the microscope in a freshly prepared mixture of an air-entraining agent and water or water and cement by fitting a glass plate at the top or side of the container.

Experiments were conducted with several proprietary agents known from previous studies of concrete to vary in their ability to produce a system of small air voids in concrete. Mixtures of (1) water and agent; (2) water, agent, and cement; or (3) lime water, sand, and agent, were shaken vigorously for 30 seconds in a 50-ml stoppered flask (mixture proportions and procedure given in captions for Fig. 4-8), and a rectangular glass cell 3 x 25 x 45 mm was filled immediately by pouring a portion of the slurry from the flask. The cell was mounted as quickly as possible on the stage of a petrographic microscope fitted with a filar ocular, a bubble was selected, and the diameter measured immediately and with passing time. Results are illustrated in Fig. 4 and 5 and are plotted in Fig. 6, 7, and 8. The abscissa at the start of each curve indicates the elapsed time between completion of agitation and first reading on the bubble.

In the absence of cement, all of the agents act similarly in that most small bubbles (less than about 100 microns in diameter) decrease in size by dissolution of the contained air, and the rate of decrease increases as the diameter of the bubble decreases (Fig. 6). With some air-entraining agents a small sac, representing the adsorbed film at the air-water interface, becomes visible just

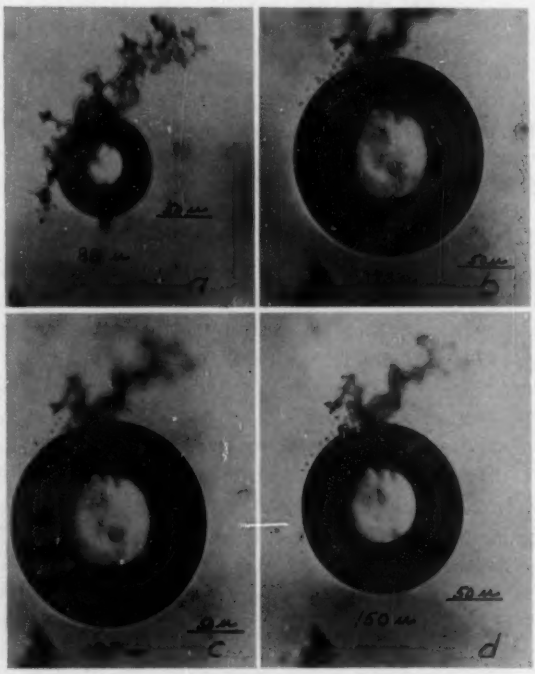


Fig. 5—Photomicrograph of an air bubble in a solution of Agent C (3 drops in 30 ml of distilled water) with a small amount of portland cement: (a) at completion of mixing, (b) after 20 min, (c) after 40 min, and (d) after 134 min. Note adhesion of flocculent cement to the bubbles

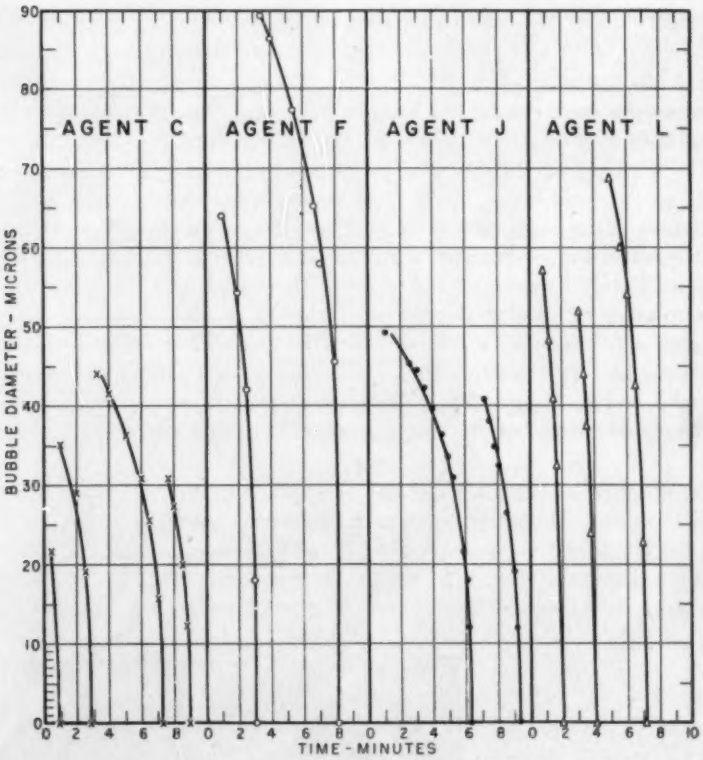


Fig. 6—Relationship of bubble size to time in water solutions of four air-entraining admixtures. Agents C and L, 3 drops in 30 ml distilled water; Agent F, 30 mg in 30 ml distilled water; Agent J, 1 drop in 30 ml distilled water. Determinations at 25-29 C

before complete dissolution of the bubble, whereupon the sac dissolves slowly and disappears. Bubbles of intermediate size, in the range of 70 to 150 microns in diameter, commonly grow at first, because they receive air from the solution while the smallest bubbles are dissolving, but later they shrink by loss of air when the smallest bubbles are largely gone (Fig. 7). In a closed cell, the large bubbles grow slowly and continuously with dissolution of the small bubbles. The history of any particular bubble is controlled by many factors other than size, such as proximity to other bubbles or the surface of the liquid, concentration of air in the water, and changes of temperature.

In the presence of portland cement or lime water, bubbles in the slurries respond differently, depending upon the composition of the agent and its concentration in the solution. Agents F and L (see Table 1) give rise to small bubbles which dissolve completely (Fig. 8). Conversely, with Agents C and J, bubbles of intermediate or small size decrease slowly and erratically in diameter and finally are stabilized for prolonged periods at various small diameters.

Larger bubbles either grow slowly or grow and then shrink to various small diameters. Again, the history of any particular bubble is controlled by many factors. As the bubbles shrink to small diameters, they become irregular in shape and appear as a crumpled sac surrounded by a dark membrane. The irregular decrease in diameter is caused by sporadic crumpling of the sac as the pressure deficiency rises with continued dissolution of air from the bubbles. At a certain diameter, the bubble becomes stabilized by increasing thickness and strength of the membrane. Some effective agents, such as sodium abietate, develop these phenomena even in very low concentrations; concentrations as low as 10 ppm have been tested with no apparent loss in effectiveness in this respect.

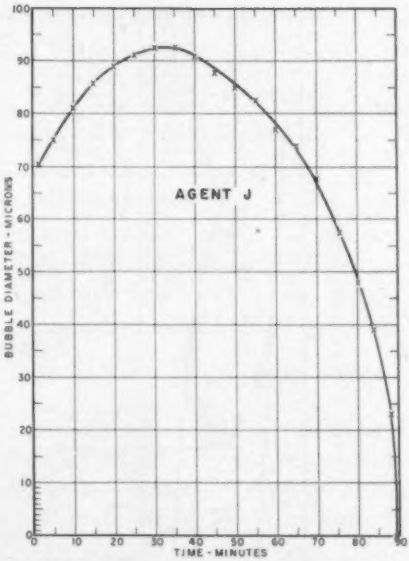


Fig. 7—Relation of bubble size to time in a water solution of Agent J. Three drops per 30 ml of distilled water at 24 C

TABLE 1—AIR-ENTRAINING ADMIXTURES INCLUDED IN THE INVESTIGATIONS

Code symbol	*Public Roads* No.*	Chemical nature	Physical state
A	—	Sodium soap of wood resins	Aqueous solution of unknown concentration
B	—	Sodium abietate	Solid or 1 percent aqueous solution
C	23	Triethanolamine salts of sulfonic acids	Designated by the manufacturer as "single strength" solution
D	8	Sodium soap of tall oil	Aqueous solution of unknown concentration
E	12	Alkyl aryl sulfonate and sodium sulfate	1 percent aqueous solution
F	—	Alkyl sulfonate	Solid
G	18	Calcium lignosulfonate with small amount of alkyl aryl sulfonate	Solid
H	—	Alkyl sulfonate	1 percent aqueous solution
I	—	Lignin sulfonic acid	1 percent aqueous solution
J	7	Sodium soaps of wood resins	5 percent aqueous solution
K	26	Alkyl aryl sulfonate and proteinaceous material	2 percent aqueous solution
L	—	Alkyl phenol of an ethylene oxide	Aqueous solution of unknown concentration

*See Jackson, F. H., and Timms, A. G., "Evaluation of Air-Entraining Admixtures for Concrete," *Public Roads*, V. 27, 1954, p. 259-277.

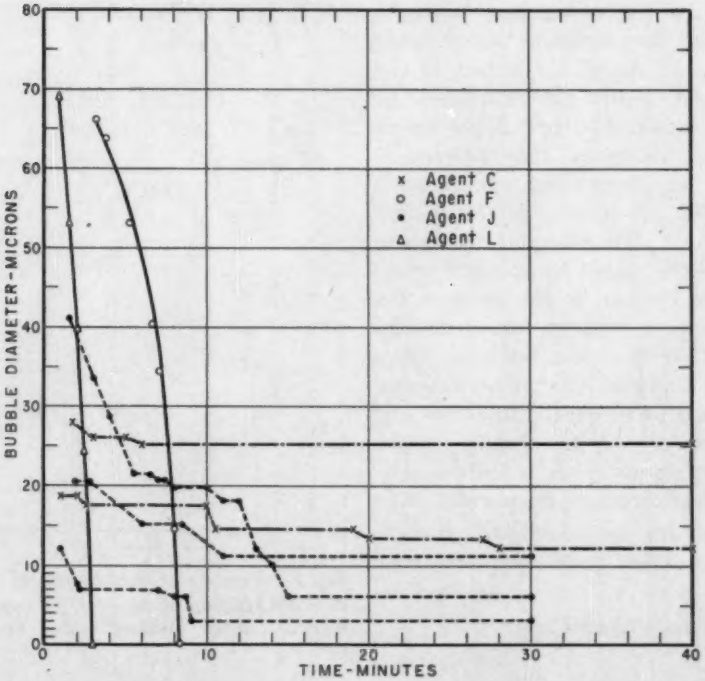


Fig. 8—Relationship of bubble size to time in water solutions of four air-entraining admixtures in the presence of portland cement (1 g per 30 ml of solution). Agents J and L, 3 drops in 30 ml distilled water; Agent F, 60 mg in 30 ml distilled water; Agent C, 1 drop in 30 ml distilled water. Determinations at 24.5-29 C

Depending upon the composition and concentration of the air-entraining agent, particles or flocculent aggregations of cement may accumulate in varying concentration at the periphery of the bubbles (Fig. 5 and 9). With some agents, such as Agent L, the bubbles are clean; with others, such as Agents A, B, C, D, and J, the bubbles typically are partially or completely plated by particles of cement or flocculent aggregations. When a coating of cement is present, it is difficult to measure the bubble diameter accurately, and it is impossible sometimes to ascertain whether the bubble has dissolved completely. Even with agents normally producing coated bubbles in this procedure, sporadic bubbles are clean and may dissolve completely. No explanation for this phenomenon is now available. With less effective agents, occasional bubbles are stabilized before complete dissolution, possibly because of contamination of the slurry by traces of previously tested better agents.

Effect of interchange of air on air content of concrete

As a result of the dissolution of air from small bubbles and its release into large bubbles, the air content of the concrete will increase unless the gain so

produced is offset by escape of air into the atmosphere from exposed surfaces of the concrete. Increase in air content ordinarily is to be expected following completion of the placing operation. For example, air entrained in bubbles 20 microns in diameter on completion of placing of the concrete will occupy about two-thirds more volume when transmitted to voids 1 mm in diameter.

Effect of interchange of air on the void system in concrete

The final result of the process of interchange is a decrease in the number of bubbles and an increase in their average size and total volume. The increase in average size is evident because, according to Willis:[8]

$$(r)_3 = \frac{3A}{4\pi M}$$

where $(r)_3$ = third moment of the radii of the bubbles, that is, the weighted average of the cubes of all the radii, A is the air content of the concrete, and M is the number of bubbles per unit volume of the concrete. Thus, the average radius will increase since A increases and M decreases by the process of interchange.

Also, since:

$$\bar{l} = \frac{4(r)_2}{3(r)_2}$$

where $\bar{l}$ = average chord length of voids on a line of microscopical traverse and $(r)_2$ = second moment of the radii of the bubbles, $\bar{l}$ must increase since the average of the cubes always will increase more rapidly than the average of the squares of the radii. Hence, the specific surface of bubbles α* will decrease inasmuch as $\alpha = 4/\bar{l}$.[1] Thus, the process of dissolution of air from and release of air into bubbles in the cement paste militates against high values of α and close spacing of the bubbles.

Hence, all factors which increase the time of setting will tend to increase the spacing of the bubbles and to decrease α. Also a high ratio of water to air will increase the rate at which small bubbles are dissolved, because more must dissolve to produce supersaturation of dissolved air in the water. For example, with a spacing factor of 0.01 in., according to Powers,† bubbles less than 3 to

*Specific surface of the void system is the area of the air-water interface per unit volume of air void and usually is expressed as in^2/in^3 or $in.^{-1}$

†Powers' formulae (Reference 1) for spacing factor of air voids in concrete, based upon microscopical linear traverse on finely ground surfaces of the hardened concrete are:

$$\bar{L} = p/4n$$

or

$$\bar{L} = \frac{3A}{4n}\left[1.4\,(p/A + 1)^{1/3} - 1\right].$$

For definition of the notations see the section on nomenclature, page 102. The first relationship is used when the air content of the concrete is high (p/A is less than 4.33), and the second is used when the air content is low.

4 microns may be dissolved completely before air begins to enter the larger bubbles. Likewise, a high water-cement ratio is conducive to a higher rate of diffusion of air through the paste because of its greater fluidity. In short, retardation of setting, high water to air ratio, high water-cement ratio, and high water content will make more difficult the production of mortar or concrete for which the spacing factor is very small.

Also, delays in placing following mixing will increase progressively the loss of air during compaction because the delay will have allowed the interchange of air to have progressed sufficiently to increase the average size and ease of escape for the bubbles. Flack[9] has shown that loss of air content from a concrete mixture originally containing 5 percent of entrained air is equivalent to approximately 0.7 percent for each hour of delay in placing.

Rate of interchange of air between air voids in unhardened concrete

No measure has been made of the rate at which air is transmitted from one bubble to another in concrete or the net rate of change effected in the parameters of the void system. An investigation of these phenomena in relation to the nature of the air-entraining admixture and properties of the mixture would be worthwhile. However, it is interesting to consider the factors which will control the rate and to review data available on bubble systems in foams and solutions.

The rate of transmission of air from small bubbles to large is directly proportional to the area of the air-water interface, the pressure differential established between bubbles losing air and those gaining air, the solubility of air in water, and the rate of diffusion of air through the water, and is inversely proportional to the distance through which the diffusion occurs.* Hence, we may write:

$$\frac{dB}{dt} \propto \frac{a\,D\,\gamma}{T}\left[\frac{1}{d_1} - \frac{1}{d_2}\right]\dagger$$

for the rate of interchange of air between the bubble of diameter d_1 and a large bubble of diameter d_2 at any time when the mass of air transmitted is B, and a = the area of the surface of the bubbles gaining or losing air (whichever is the smaller); D = a diffusion factor which increases with the solubility of air in the water, the rate of diffusion of molecules of the gas through the water, and the permeability of the adsorbed film at the air-water interface for air; γ = surface tension at the air-water interface; and T = the average distance between the surface of the bubbles. For a system of entrained air bubbles, the effective surface area through which the transmission takes place depends upon the size distribution of the bubbles and should vary directly with the specific surface α, and the distance between the surface of the bubbles should vary directly with the spacing factor $\overline{L}$.

*The rate of transmission will depend also upon the permeability of the film at the air-water interface with respect to the constituents of air. The properties of the film depend upon the characteristics of the air-entraining agent, its concentration, and the area of the air-water interface relative to the volume of the water phase.

†Compare the derivations by Clark and Blackman (Reference 11, p. 3) and by Bikerman (Reference 10, p. 120).

Study of the water-air foams demonstrates that the surface area of bubbles decreases and the average diameter of bubbles increases greatly during the first few minutes after the foam is formed.[11] For example, between 1½ and 8 min after formation of a foam in a given 5 percent soap dispersion, the number of bubbles about 25 microns in diameter was decreased by about two-thirds, and the number of voids about 150 microns in diameter was decreased by a factor of about 25, while the number of bubbles above 500 microns in diameter was increased somewhat. Also, in a foam produced in a 5 percent solution of an aluminum protein hydrolysate, Clark and Blackman[11] observed a decrease in the specific surface from 660 to 225 in.$^{-1}$ during 1 hr.

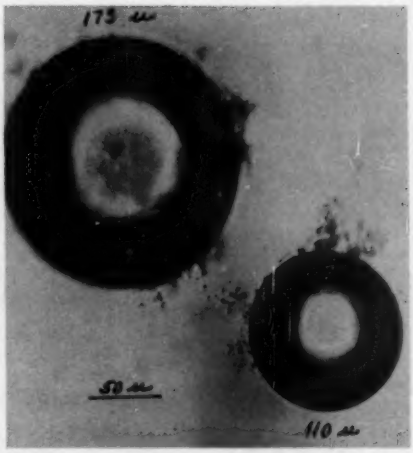


Fig. 9—Photomicrograph of two air bubbles in a dilute slurry of portland cement and a solution containing 3 drops of Agent J in 30 ml distilled water. Note adhesion of flocculent cement to the bubbles

The rate of transmission of air from bubble to bubble in concrete undoubtedly is less than for air-water foams, because the distance from bubble to bubble is greater; solubility of the air in the water phase is less than in pure water because of the electrolytes in solution; and diffusion of the dissolved air probably is decreased by the high viscosity of the water adjacent to particles of hydrating cement. Nevertheless, there is no reason to believe that the process is not significant in modifying the parameters of the void system in cement paste.

In an infinite nonsetting mass of water and air bubbles, the air would finally accumulate in relatively few large bubbles, which are in equilibrium with the air dissolved in the water and with each other under the conditions of temperature and pressure prevailing in the mass. The rate of interchange will become very small when the diameter of the voids is greater than 100 microns, because the difference in internal pressure is very slight.

Influence of water-cement ratio on bubble formation and change in neat cement slurries

For any given portland cement and proportion of an air-entraining admixture, there will be an optimum water-cement ratio for neat pastes or slurries at which a maximum frequency (minimum spacing) of voids will develop and be retained after hardening. At water-cement ratios in the range producing stiff but plastic consistency in the mixture (e.g. at water-cement ratio about 0.3 by weight), water films on the cement will be insufficient to produce an adequate foaming action, in-folding of air during mixing will be minimized, and thorough

TABLE 2—INFLUENCE OF WATER-CEMENT RATIO ON THE AIR VOID SYSTEM OF NEAT CEMENT PASTE*

Mix series	Water-cement ratio by weight	Air content by pressure meter, percent†	Parameters determined microscopically					
			Air content, percent	Average chord intercept, in.	Specific surface of voids, in.$^{-1}$	Number voids per in. of traverse	Paste to air ratio	Spacing factor, in.
V-5	0.30	3.0	4.13	0.0042	950	9.612	24.21	0.0098
V-6	0.40	4.0	5.59	0.0030	1330	18.629	17.89	0.0063
V-7	0.50	4.5	6.93	0.0039	1030	17.775	14.43	0.0073
V-8	0.60	3.3	8.77	0.0052	770	16.860	11.40	0.0087

*Cement No. 9406 (Na_2O = 0.18 percent; K_2O = 0.03 percent). Agent J, 1.33 ml per lb of cement; 0.4 cu ft batch mixed 4 min in 3.5 cu ft capacity drum-type mixer; water temperature = 70 F; 3 x 6-in. cylinders vibrated 2 sec in each of two lifts; specimens stored 4 days in moist room at 73.4 ± 3.0 F before preparation of ground surfaces for microscopical determinations.
†In accordance with ASTM C 231.

compaction of the paste may be difficult. For such a mixture, air content will be lower and the voids will be larger than would be produced with the same proportion of air-entraining admixture at higher water-cement ratio. At intermediate water-cement ratio (e.g., 0.4 to 0.6 by weight), abundant air bubbles will be produced in the presence of an adequate proportion of an air-entraining admixture, but the average size of the bubbles will increase as a result of interchange of air between small bubbles and large, the interchange increasing with the water-cement ratio. At excessive water-cement ratio, a large proportion of the bubbles will be lost by progressive enlargement and escape from the mixture.

These relationships are illustrated in Table 2. Four 0.4 cu ft batches of cement, water, and a solution of Agent J, at the rate of 1.33 ml per lb of cement, were mixed in a drum-type mixer and cast into 3 x 6-in. cylinders and vibrated 2 sec internally in two lifts. The water-cement ratio ranged from 0.3 to 0.6 by weight. The air content was determined in accordance with ASTM C 231[13] in a pressure meter at the time the specimens were prepared. The specimens were cured 4 days in a moist room at 73.4 ± 3.0 F. Sawed surfaces were later prepared for microscopical determination of the parameters of the void system by the linear traverse method.*

The air content determined on the unhardened mixture is materially lower than that determined microscopically on the hardened specimens, and the difference between these values increases progressively with water-cement ratio. The mixture with water-cement ratio 0.3 apparently contains insufficient water to permit adequate foaming and in-folding of air during mixing. The air content is low, and the spacing factor is high. Following placement, the air content increased 1.1 percent as a result of interchange of air. Water-cement ratio of 0.4 is optimum in this series, based upon the minimum value of spacing factor. Surface area and frequency of the voids are greatest at this water-cement ratio. Air content increased 1.6 percent before hardening of the specimen. The spacing factor increases progressively with increase of the

*The general procedures employed and nomenclature applied are those developed by Brown and Pierson,[12] Powers,[1] and Willis.[6] Details of procedures developed by the authors are described in the appendix to Part 4.

TABLE 3—INFLUENCE OF ALKALI CONTENT OF MIX ON THE AIR VOID SYSTEM OF NEAT CEMENT PASTE*

Mix series	Water-cement ratio, by weight	Air content by pressure meter, percent†	Parameters determined microscopically					
			Air content, percent	Average chord intercept, in.	Specific surface of voids, in.$^{-1}$	Number voids per in. of traverse	Paste to air ratio	Spacing factor, in.
V-9	0.30	2.8	4.76	0.0046	870	10.170	21.00	0.0100
V-10	0.40	4.0	6.79	0.0051	780	13.425	14.73	0.0095
V-11	0.50	4.3	6.66	0.0061	660	10.861	15.01	0.0115
V-12	0.60	3.3	7.38	0.0070	570	10.530	13.55	0.0127

*Procedure and materials identical with those stipulated in footnotes to Table 2, except for addition of sodium hydroxide (NaOH) at the rate of 6.0 g per lb of Cement No. 9406.
†In accordance with ASTM C 231.

water-cement ratio to 0.5 and 0.6, and the increase in air content after placing is likewise increased to approximately 2.4 and 5.5 percent, respectively.

Influence of alkali content of the cement on bubble formation and change in neat cement slurries

Many air-entraining agents produce, at air-water interfaces in the presence of calcium ions, a film composed primarily of the calcium salt of the surface active constituent of the agent. This film may vary in such properties as permeability for air, strength, and elasticity, with changes in composition and thickness. Because high concentrations of alkalies (Na^+ and K^+) in solutions such as are developed during the hydration of portland cement strongly depress the concentration of calcium in solution, it is expected that cements which provide an immediate high concentration of sodium or potassium ions in solution will cause production at air-water interfaces of precipitated films which are thinner and more soluble than those which would develop in the presence of low-alkali cements.

A series of air-entrained neat cement mixtures was prepared, using materials and procedures identical with those described in Table 2 and in the text of the previous section, except that sufficient sodium hydroxide was added in the mixing water to produce a concentration of sodium in the mixture equal to that which would be produced by a cement containing 1.20 percent Na_2O, assuming that all of the sodium goes into solution at once.

The trend of results (Table 3) corroborates the conclusions previously stated concerning the significance of water-cement ratio. However, there was a consistent increase in the spacing factor, and a consistent decrease in the surface area of the voids determined in the hardened specimen in comparison with the equivalent specimens containing the same cement but no added sodium hydroxide. The proportion of air entrained in the mixing process and remaining after placement of the mixture in the air meter is essentially identical with that in the equivalent specimens without the added sodium hydroxide. Hence, the sodium hydroxide appears to have decreased the degree to which the air content is preserved in small bubbles. The dissolution of the small bubbles proceeds both before and after the determination of air content by the pressure meter.

Influence of composition of air-entraining admixtures on bubble formation and change in neat cement slurries

To determine the ability of several air-entraining admixtures to entrain air in portland cement paste without the entrapping influence of aggregate, a series of mixtures was prepared to simulate the proportions of the cement, water, and admixture previously found necessary to entrain 5 percent of air in concrete containing ¾-in. maximum-sized natural aggregate. Mixing time was 4 min in a 3½-cu ft revolving drum mixer. Air content was determined in a pressure meter in accordance with ASTM C 231;[13] 3 x 6-in. cylinders were cast, using 2 sec internal vibration in each of two lifts. The specimens were cured 4 days at 73.4 ± 3.0 F in a moist room, after which finely ground surfaces were prepared for microscopical traverse. Results are summarized in Table 4.

The neat cement specimens contained more air than might have been anticipated, considering the fluid character of the mixture and the vibration given each specimen during placing. For the two superior agents (C and J), the air content of the neat pastes is approximately 43 percent of the air content of the comparable concrete. From this, it might be concluded that more than half of the air content of the concrete is retained by inclusion of the bubbles among the aggregate. For these agents, the determined values of average void intercept are virtually identical, and the values of α (specific surface) are the same or similar. The spacing factor for the neat specimen is somewhat higher than

TABLE 4—INFLUENCE OF AIR-ENTRAINING ADMIXTURES ON THE AIR VOID SYSTEM OF CEMENT PASTE*

Agent	Mix	Air content by pressure meter, percent†	Parameters determined microscopically					
			Air content, percent	Average chord intercept, in.	Specific surface of voids, in.$^{-1}$	Number voids per in. of traverse	Paste to air ratio	Spacing factor, in.
J	Neat cement	4.8	6.98	0.0038	1050	18.322	14.33	0.0069
	Concrete	5.1	3.96	0.0038	1050	39.809	6.62	0.0051
C	Neat cement	5.0	7.77	0.0041	980	18.688	12.87	0.0073
	Concrete	5.1	4.56	0.0045	890	38.702	5.75	0.0058
F	Neat cement	3.9	5.57	0.0046	870	12.038	17.95	0.0093
	Concrete	4.6	5.17	0.0061	660	32.218	5.09	0.0071
L	Neat cement	4.4	6.76	0.0093	430	7.266	14.79	0.0176
	Concrete	4.8	2.30	0.0084	480	10.873	10.96	0.0140
None	Neat cement	0.9	1.70	0.0051	780	3.303	58.82	0.0169

*A series of concrete mixtures was prepared using various air-entraining admixtures in proportions to produce an air content approximating 5 percent; water-cement ratio was constant at 0.45; slump was 3 ± ½ in.; ¾-in. maximum aggregate; 40.12 percent sand in the aggregate; 241-266 lb of water per cu yd; paste content equals 24.3 to 26.3 percent by volume. Air void parameters of the concrete were determined microscopically. Subsequently, neat cement mixtures were prepared to simulate the concrete containing each of the 4 admixtures, all factors and procedures being held constant except that no coarse or fine aggregate was included. Air void parameters were determined microscopically on these specimens and on a neat paste containing no air-entraining admixture.

that for the concrete, primarily because the number of voids per inch of traverse in the neat specimens is only about 45 percent of that in the pastes of the comparable concretes.

Agent F, an alkyl sulfonate, is of intermediate quality as an air-entraining agent. The bubble size and frequency are consistently less, and the spacing factor is consistently greater with this agent than with either Agent C or J. The neat specimens contain only 26 percent as much air as does the paste of the comparable concrete.

Agent L, a relatively ineffective air-entraining agent, produces equivalent void systems in the neat paste and the paste of the comparable concrete, with respect to void size, frequency, and spacing. The indicated air content of the hardened neat paste is very high in comparison with that of the comparable concrete. This relationship is anomalous in view of the concepts discussed above, since one would expect a lower ratio of air content of the neat specimen to that of the equivalent concrete for Agent L than for Agents C, F, or J. Possibly the answer lies in the unusually low air content of the concrete containing Agent L. Other concrete specimens containing Agent L show normal air content in spite of very high values for spacing factor. Since the average air bubble in concrete containing Agent L is large, the air is very susceptible to loss during placing. Possibly a slight excess of vibration of the concrete containing Agent L accounts for the low air content of the hardened concrete.

EFFECT OF ENTRAINED AIR ON WORKABILITY OF CONCRETE

As is well known, entrainment of air in fresh concrete by means of an air-entraining agent improves workability enough that the water content and sand content of the mixture can be reduced significantly. Experience shows that the sand content can be reduced by an amount approximately equal to the volume of the air entrained, and that the water content can be reduced 2 to 4 percent for each 1 percent of air entrained, without loss in slump. It is interesting to consider the role of the entrained air bubbles in this action in relation to data developed in this study.

For adequate workability of concrete, aggregate particles must be spaced so that they can move past one another with comparative ease during mixing and placing. In non-air-entrained concrete, workability can be achieved by including sufficient fine sand, cement, and water to separate the particles of coarser aggregate and supply a matrix in which movement can occur with a minimum of mutual interference. By such means, spacing of the solids is increased and the dilatancy necessary to manipulation of the fresh concrete is reduced, with consequent reduction of work required. To select two schemes of proportioning among many, we note those developed by Weymouth[14] and C. T. Kennedy[15] to control workability.

In Weymouth's scheme,[14] grading of the aggregate is calculated so that each successive size fraction is spaced in the fresh concrete so as to permit passage of all smaller sizes. This requires more pea gravel than ordinarily is used and,

if the grading is calculated into the sand range, the proportion of sand is increased greatly also. Conversely, C. T. Kennedy[15] proposed a procedure of mixture proportioning in which the desired spacing and minimum particle interference could be achieved by selection of a percentage of sand giving a minimum requirement for the volume of cement paste necessary to fill interparticle voids and to coat each particle of sand and coarse aggregate with a film of a thickness found necessary to produce the desired slump at a selected water-cement ratio.

Proportioning of concrete mixtures involves a compromise between requirements for workability and requirements for strength, durability, volume stability, and other properties of the hardened concrete. Workability requires that the interparticle voids in the aggregate be more than filled by cement paste, whereas optimum qualities of the hardened concrete require that these voids be just filled, or for most purposes not completely filled, the latter being true for all concrete exposed to freezing and thawing.

It is the role of entrained air to resolve this conflict. First, entrained air voids increase the effective volume of the cement paste during mixing and placing, and so eliminate the need for that portion of the paste content added specifically to induce workability. Kennedy calculates this portion of the cement paste as 20 to 35 percent of the total paste content of the concrete. Second, since almost all of the volume of entrained air is in air voids of sand size, they may be considered to increase the effective volume of mortar during mixing and placing and so to compensate for lack of sand. The density desired in the placed concrete is obtained by compaction, as a result of which one-half to two-thirds of the air content of the fresh concrete is driven out.

For example, for air-entrained concrete, the sand percentage selected for routine test mixtures in Bureau of Reclamation laboratories approximates that giving the maximum weight of aggregate per unit volume of concrete. For the particular ¾-in. maximum aggregate used, the dry-rodded sand and gravel include interparticle voids amounting to 24.9 percent of the total volume. The paste content of a specimen of the placed concrete with a water-cement ratio of 0.51 by weight is 26.8 percent, and the air content is about 6.7 percent. Using similarly graded gravel and sand, but varying sand percentages, the paste content derived by Kennedy's method is 32.1 percent at the sand percentage used in the actual mix (45 percent by weight) and attains a minimum of 31.4 percent when the sand constitutes 30 percent by weight of the entire aggregate. Assuming that entrapped air accounts for 2 percent of the concrete, the paste plus air content of the concrete proportioned according to Kennedy is about 34 percent, in contrast to 33.5 percent for the entrained-air concrete. This close correspondence exists in spite of a reduction of 0.82 bag of cement per cu yd of concrete in conjunction with the introduction of entrained air.

Calculation in accordance with Weymouth indicates that optimum workability would be obtained with the aggregate ordinarily used in USBR laboratories if sand constitutes 60 percent by weight of the total aggregate in contrast

to the amount (45 percent) found to yield maximum dry-rodded unit weight for ¾-in. maximum aggregate. Evidently, the workability inherent in the Weymouth grading can be achieved if the fresh concrete were to contain a sufficient number of air voids in the sand size range to compensate for the deficiency in sand content. In other words, if the laboratory mixture were to contain about 15 percent of air while being mixed and placed, the Weymouth grading would be approximated even though the sand percentage is about 45.

Concrete allowed to harden after being taken from the mixer, but not molded or compacted, reveals air content as high as 13.5 percent; for the same concrete air content is 6.5 percent after being molded into a 3 x 6-in. cylinder. Hence, the air content of this concrete can be assumed to be 15 percent or more while being worked since loss of air will occur as a result of settlement after mixing, even without molding.

Thus, it appears that entrained air produces in concrete during mixing and placing a mortar content approaching that required by the Weymouth grading, but the benefits of a coarser grading of the aggregate are obtained as a result of loss of a large proportion of the original air content during compaction. Calculation indicates that the air content of fresh concrete necessary to simulate a Weymouth grading of the aggregate ranges from about 5 percent for concrete containing 6-in. maximum aggregate to about 16 percent for ⅜-in. maximum aggregate. From 40 to 65 percent of this air is lost during compaction in achieving the range of air content recommended for air-entrained concrete by ACI 613-54.[2]

EFFECT OF ENTRAINED AIR ON SEGREGATION AND BLEEDING OF CONCRETE

Segregation and bleeding of concrete are differing manifestations of loss of the homogeneity produced by adequate mixing. Segregation usually implies separation of coarser aggregate from the mortar or separation of cement paste from aggregate. Bleeding is the autogenous flow of mixing water within, or its emergence from freshly placed concrete, usually as a result of sedimentation of the solids through the water with the resulting appearance of a layer of water at the surface or as a result of drainage of water laterally or downwardly from the concrete.

Segregation and bleeding are reduced greatly by air entrainment adequate to control or significantly improve resistance of the concrete to freezing and thawing. These actions probably result primarily from four physical and physical-chemical phenomena related directly to the incorporation of a system of small bubbles in the fresh concrete. First, the bubbles buoy up the aggregate and cement and so reduce the rate at which sedimentation occurs in the placed concrete. Second, the bubbles decrease the effective area through which the differential movement of water may occur. Third, adhesion between the cement and aggregate is augmented to the extent that mutual adhesion is established between the bubbles and particles of cement and aggregate. Fourth, the

surface area of voids in the plastic concrete is sufficiently large to retard the rate at which water will separate from the paste by drainage.

Powers[16] has discussed the effects of buoyancy and decreased cross-sectional area of the capillary passages through which water moves during bleeding. He concluded that the combined effects of these phenomena are a complex function showing a decrease in bleeding rate resulting from sedimentation as the air content of the paste increases. The force of buoyancy may be visualized as operating against the entire granular structure of the aggregate and cement and also with respect to individual particles of cement and aggregate adhering to the bubbles.

Adhesion between the bubbles and solid particles in the mixture and the surface area of the voids act together to increase the stiffness of the paste. As has been stated previously, the force of adhesion between bubbles and cement, or bubbles and specific minerals in aggregates, cannot be evaluated from data now available. Nevertheless, existence of these adhesive forces is indicated qualitatively by the observation of flotation of cement and aggregate in water slurries of various air-entraining agents and measurement of finite contact angles between cement clinker and air-water interfaces in the presence of air-entraining admixtures.

Entrainment of air in concrete increases the internal surface area (that is, air-water, air-solid, and water-solid interfaces) of a given mass of cement paste and so increases capillary compression of the concrete and decreases the thickness of water films over the internal surfaces. Of course, the unit weight of the concrete, and especially of the paste, is decreased. These actions combine to decrease separation of cement paste from aggregate because the paste is made more viscous by resistance of capillary forces at the external surface to increase of surface area of the mass.

For concrete proportioned to contain 5 or 6 percent of air after placing, the air content of the paste while being worked may be in the range from 40 to 60 percent. Hence, air contents considerably above that specified for the placed concrete are present to improve the resistance of fresh concrete to segregation. Data to be presented subsequently will show that well-air-entrained concrete containing ¾-in. maximum aggregate has about 2 to 7 million voids per cu in. of concrete after placing and hardening. The cement paste in such concrete contains 7 to 24 million voids per cu in. of paste. No method has been developed for measuring the number of bubbles existing in the unhardened cement paste.

The voids still present in the hardened paste present an air-water interface as much as 233 sq in. per cu in. of cement paste; the interface area is undoubtedly several times this value in the equivalent fresh concrete. This value is equivalent to that of foams in 5 percent solution of protein hydrolysate approximately one-half hour after their formation.[11]

Hence, the bubble system produced in well-air-entrained cement paste is analogous in frequency and size of bubbles to foams produced in aqueous solutions, and they should respond similarly to forces of gravity and manipulation. The properties of the air-entrained paste as a structural mass (apart from

the history of individual bubbles or changes of specific surface resulting from transfer of air from small bubbles to large) depend primarily upon capillary forces. Capillary forces compress the foam, resist increase in external surface area of the mass, and hold within the foam excesses of liquid against gravity. With loss of air and decrease of surface area of bubbles contained in a unit volume of foam, the capillary forces decrease. A satisfactory air-entraining agent retards deterioration of the foam included in the cement paste sufficiently, during handling and placing and until hardening occurs, to maintain these forces at levels adequate for workability and minimal bleeding.

CONCLUSIONS

1. Air in concrete arises in several ways and is entrapped in voids of two general types: (1) those subjected only to capillary and hydraulic pressure of water in the fresh concrete; and (2) those subjected to load of both water and solids. These are commonly called "entrapped" voids and "entrained" air bubbles, respectively. The "entrapped" voids are ineffective or harmful in unhardened concrete because they do not increase the spacing of solid particles in the mass and hence they do not decrease the dilatancy necessary to manipulation. Entrained air bubbles increase the spacing of the solid particles and decrease the dilatancy; moreover, they facilitate movement of particles of aggregate by separating solid surfaces and sustaining short-time loads.

2. The surface-active constituents of air-entraining agents are positively absorbed at air-water interfaces, causing a decrease of surface tension and producing a film whose properties differ from those of the bulk of the mixing water. Several effective air-entraining agents produce films composed of the calcium salt of the anion of the surface-active constituent at air-water interfaces, water-cement contacts, and probably at certain of the water-aggregate contacts.

3. Air is interchanged continuously in unhardened concrete between small bubbles and large as a result of the greater excess pressure maintained in small bubbles. The result of such interchange is an increase in air content of the concrete and a decrease in the specific surface of the bubbles. The rate of such interchange in concrete is not established.

4. Adsorbed films at air-water interfaces produced by satisfactory air-entraining agents decrease the rate of transfer of air, decrease the tendency of bubbles to coalesce, permit bubbles to bear short-time loads, and decrease the work necessary to produce bubbles of given specific surface.

5. Precipitation or adsorption of the air-entraining agent's surface-active constituent (or its anion) at the surface of cement or aggregate particles decreases the hydrophilic quality of these surfaces and increases the force of adherence of air.

6. Formation of an optimum void system in fresh concrete is influenced by several factors: (1) reduction in surface tension by the concentration of air-entraining agent used; (2) consistency of the mixture; (3) duration and vigor

of mixing; (4) water-cement ratio and water content of the mixture; and (5) grading of the aggregate plus cement.

7. The rate and degree of deterioration of the bubble system in fresh (unhardened) concrete is increased by many factors, most important of which are: (1) ineffectiveness of the air-entraining agent in retardation of passage of air across an air-water interface, and in establishment of adhesion between air bubbles and cement or aggregate or both; (2) variation from optimum concentration of air-entraining agent; (3) increase in water-cement ratio; (4) increase of air-water ratio; (5) retardation of setting of cement; (6) delays in placing; and (7) increased alkali content of the cement.

8. Air entrainment improves workability by decreasing dilatancy of mixture for given content of cement paste (cement plus water), with a resulting decrease in the necessary manipulation. Removal of air during compaction produces hardened concrete considerably more dense than the concrete being worked. Such change of density from the optimum for working to a density approaching optimum for stability of hardened concrete cannot be achieved by use of excess cement, fines in the aggregate, or powdered admixtures.

9. Air entrainment decreases segregation and bleeding by imparting a buoyant action to cement and aggregate, decreasing the area otherwise available for differential movement of water and solids, and producing in the cement paste being worked a foam in which capillary forces develop stiffness and retard drainage of mixing water.

10. Much additional work is justified to better understand the physical, physical-chemical, and chemical processes involved in the production of consistently satisfactory and uniform air-entrained concrete.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to colleagues in the Engineering Laboratories, Bureau of Reclamation, Denver, for generous cooperation and assistance. In particular, they recognize the contribution of E. J. Benton, R. F. Adams, E. L. Ore, and A. B. Crosby. The photomicrographs were prepared by W. M. Batts. Much valuable criticism of the manuscript was received from W. H. Price, W. T. Moran, and L. H. Tuthill.

REFERENCES

1. Powers, T. C. "The Air Requirement of Frost-Resistant Concrete," *Proceedings*, Highway Research Board, V. 29, 1949, pp. 184-202; *Bulletin* No. 33, Portland Cement Association, pp. 1-19.

2. ACI Committee 613, "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," ACI JOURNAL, Sept. 1954, *Proc.* V. 51, pp. 49-64.

3. Kennedy, H. L., "Entrained Air—Its Effect on the Constituents of Portland Cement Concrete," *Proceedings*, ASTM, V. 44, 1944, pp. 821-830.

4. Walker, Stanton, and Bloem, D. L., "Studies of Concrete Containing Entrained Air," ACI JOURNAL, June 1946, *Proc.* V. 42, pp. 629-640.

5. Taggart, A. F., *Handbook of Mineral Dressing*, John Wiley and Son, New York, 1945, pp. 12-37.

6. Gaudin, A. M., *Principles of Mineral Dressing*, McGraw-Hill Book Co., Inc., New York, First Edition, 1939, 554 pp.

7. *Handbook of Chemistry and Physics*, Chemical Rubber Publishing Co., Cleveland, 30th Edition, 1948, p. 358.

8. Willis, T. F., Discussion of "The Air Requirement of Frost-Resistant Concrete," *Proceedings*, Highway Research Board, V. 29, 1949, pp. 203-211; *Bulletin* No. 33, Portland Cement Association, pp. 20-28.

9. Flack, H. L., "Loss of Entrained Air in Freshly Mixed Concrete and the effect of This Loss on the Properties of the Hardened Concrete," *Materials Laboratory Report* C-431, Bureau of Reclamation, 1949, 6 pp.

10. Bikerman, J. J., *Foams: Theory and Industrial Applications*, Reinhold Publishing Corp., New York, First Edition, 1953, 345 pp.

11. Clark, N. O., and Blackman, M., "The Degree of Dispersion of the Gas Phase in Foam," *Transactions*, Faraday Society, V. 44, 1946, pp. 1-7.

12. Brown, L. S., and Pierson, C. U., "Linear Traverse Technique for Measurement of Air in Hardened Concrete," ACI JOURNAL, Oct. 1950, *Proc.* V. 47, pp. 117-124.

13. *ASTM Book of Standards, Part 3*, American Society for Testing Materials, 1955, pp. 1287-1293.

14. Weymouth, C. A. G., "Designing Workable Concrete," *Engineering-News Record*, Dec. 29, 1938, p. 818.

15. Kennedy, C. T., "The Design of Concrete Mixes," ACI JOURNAL, Feb. 1940, *Proc.* V. 36, pp. 373-400.

16. Powers, T. C., "The Bleeding of Portland Cement Paste, Mortar and Concrete," *Bulletin* No. 2, Portland Cement Association, 1939, 160 pp.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

Load Test on Flat Slab Floor with Embedded Steel Grillage Caps*

By DONALD D. MEISEL†, CRYIL D. JENSEN‡, and WALTER H. WHEELER§

SYNOPSIS

Steel grillage caps embedded in concrete flat slab and attached to steel columns were used in the addition to Smith, Kline, and French Laboratories. Because this was the first appearance of such a design in Philadelphia, a full scale load test was performed on the second floor of the new addition.

Since the second floor was designed for movable load of 300 psf, the static test load was doubled to 600 psf. Bags of rock salt (1,500,000 lb total) were spread over 2500 sq ft, and shifted to alternate locations. Stresses were determined by use of SR-4 strain gages fastened to the reinforcing steel and concrete, and dial micrometers installed at critical points under the slab measured deflections.

INTRODUCTION

The first to sixth floor slabs of Building D, Smith, Kline, and French Laboratories, Philadelphia, are flat slab, "Smooth Ceilings"** system with steel grillage caps (Fig. 1) embedded in the slabs and attached to the steel columns. The caps are so named because they function in floor slabs as inverted grillages and when used in mat foundations as grillages. They are designed as such with the arms acting as uniformly loaded cantilevers projecting from the columns.

The Philadelphia Building Code did not give credit to the grillages in bending, and the structural engineers designed the slabs accordingly. However, analysis of the slab designs indicated that if the grillages could be considered equivalent to concrete capitals of equal diameter in reducing bending moments, 35 to 40 percent of the slab reinforcement could be omitted.

R. B. Ferguson, then chief engineer of Smith, Kline, and French Laboratories, directed that the matter be taken up with the Philadelphia Building Department. An agreement was made whereby the floor slabs could be redesigned treating the embedded steel grillages as equal to a concrete capital and using the empirical total moments of the 1951 ACI Code as the total slab moments for elastic analysis. It was stipulated that a load test be made on the second

*Received by the Institute Mar. 19, 1957. Title No. 55-6 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Member American Concrete Institute, Chief Engineer, E. L. Conwell and Co., Philadelphia, Pa.

‡Professor of Civil Engineering, Lehigh University, Bethlehem, Pa.

§Member American Concrete Institute, Consulting Engineer, Minneapolis, Minn.

**The trade name "Smooth Ceilings" was adopted because, normally, these ceilings are all flat and smooth. Drop panels are sometimes used, as well as waffle type ceilings. Trademark has been applied for. Copyrights 56,955; 50,114; 118,712; 148,724; 170,707; and 181,774 were granted. U. S. Patents No. 1950422 and 2000543 and Canadian Patent No. 351,321 have been issued.

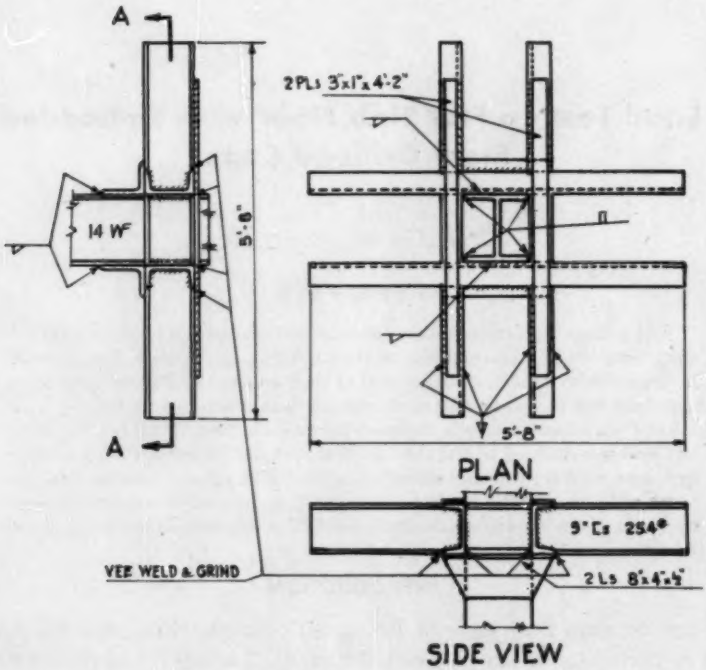


Fig. 1—Grillage cap details

floor slab of the building in accordance with the following specifications provided by the building department. The test load was to be twice the design live load, plus any superimposed dead load added to the slab, such as finish. Deflection and recovery of the slab were to be judged by the 1951 ACI Code. If the slab met the test it would be rated at full capacity. It it did not meet the test it would be rated for the loading at which it did meet the test requirements of deflection and recovery. The load was to be applied in six increments.

The second to sixth floor slabs were designed as stated, except the total empirical positive moments were increased by 10 percent before distribution to compensate for the redistribution of stress which occurs as the concrete shrinks or flows during the first year or two after construction. The first floor slab was not redesigned because construction had so progressed that it might delay completion of the building. The second floor slab (Fig. 2) was designed for uniform live load of 300 psf and was designated for the test. Fig. 3 shows the slab reinforcement. The slab is 14 in. thick with 1 in. of monolithic finish, and assumed equal to 14½ in. thick for design purposes. The steel columns which support the test portion of the slab are all 14 WF 320 with flange plates.

The building has been extended to 12 stories since the test was made, using light steel construction for the upper six stories.

The E. L. Conwell Co. was employed by the owner to arrange for and run the test in accordance with the specifications of the Philadelphia Building Department. Prof. Paul Andersen, University of Minnesota, was employed to collaborate, and consulted with the company regarding test arrangements. He was called to Korea before the test was started and Professor Jensen was then employed to collaborate in running the test, compiling the data, and preparing the report. The maximum deflection in the six panels that were loaded was 0.3115 in. The 1956 ACI Code permits 0.517 in., and requires 75 percent recovery 24 hr after load is removed. Recovery in this case was about 87 percent of the 0.3115 in. deflection. Details of the test follow.

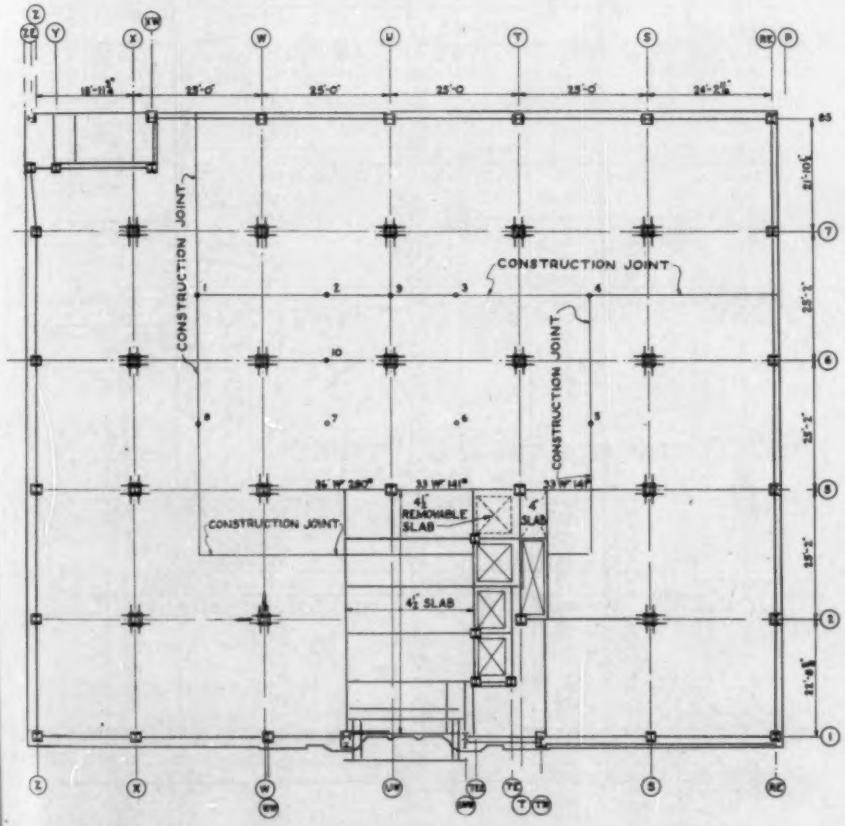


Fig. 2—Location plan, second floor, Building D of Smith, Kline, and French Laboratories

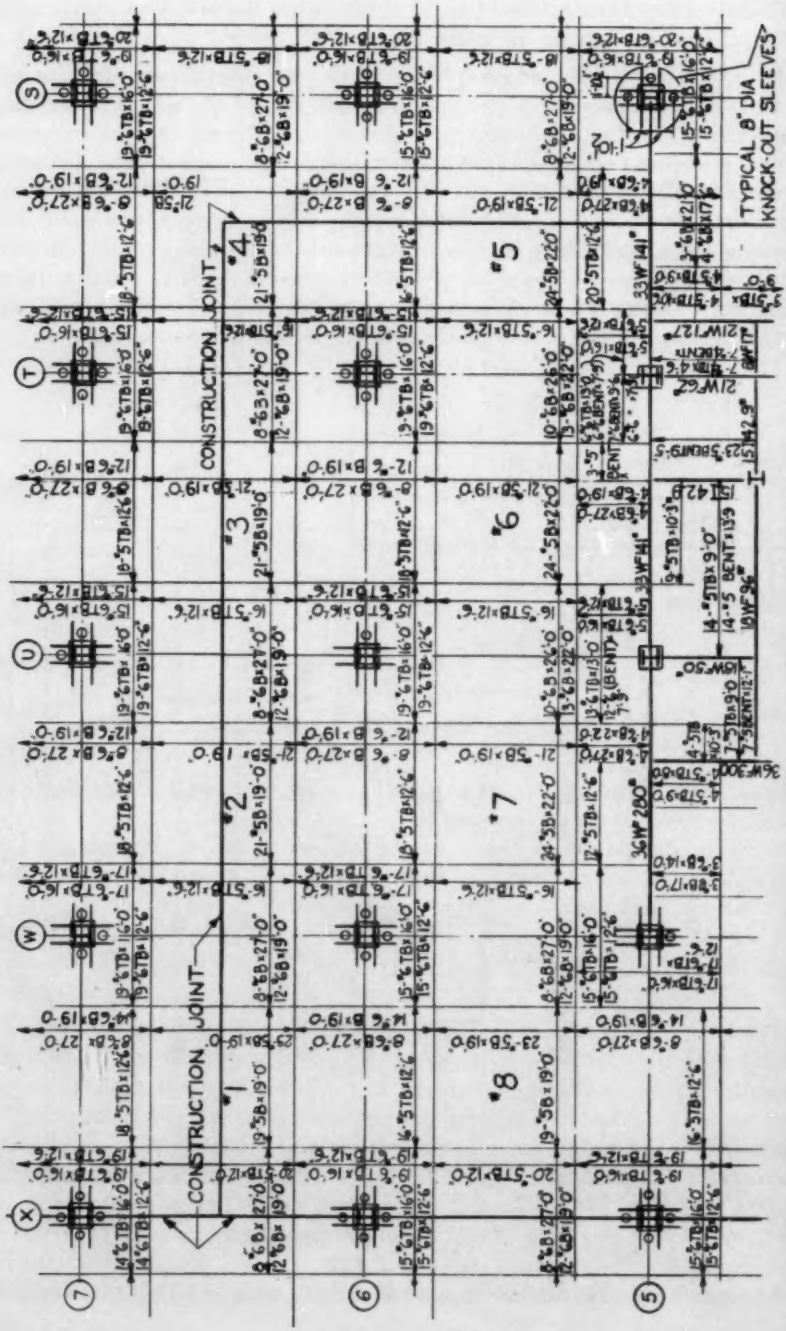


Fig. 3—Floor slab reinforcement schedule

TEST PROCEDURE

The load test was begun Aug. 22, 1956 after installation of the strain and deflection gages; a protective shoring safeguarded the construction personnel who continued work on the floor below the test panels.

Installation of strain gages

SR-4 electrical resistance strain gages, size A-1, were used. They required a space approximately 2 x ½ in. and were attached to the concrete surfaces and the surfaces of steel flanges of channels and reinforcing bars. In the case of steel reinforcing bars, it was necessary to remove (by filing or grinding) all corrugations for a distance of about 3 in., and the gages were formed to fit the curvature of the bars.

Casting of the slab was completed before the gages were installed. Fig. 4 shows wooden blocks over and under reinforcing bars and steel channels which were withdrawn after the concrete was placed, giving easy access to places of attachment.

In order to prevent any direct contact between gages and the salt which was used for loading, where strain gages were attached to the top surface of the concrete slab and at the bottom of slots, small inverted boxes were made and placed over them. Both boxes and covers had holes for the connecting wires which ran through metal conduit beneath the applied loads to the SR-4 apparatus.

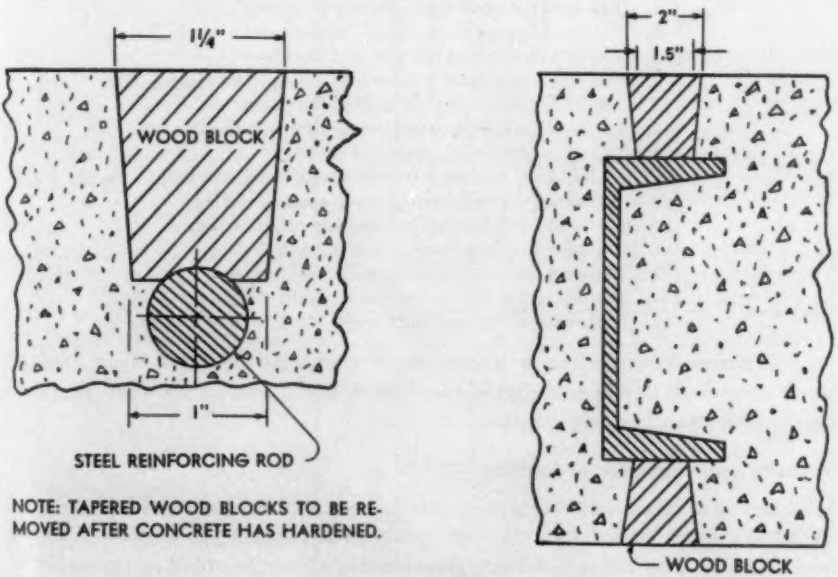


Fig. 4—Removable wood blocks allowed SR-4 gages to be attached to channels and reinforcing steel after concrete had hardened

Fig. 3—Floor slab reinforcement schedule

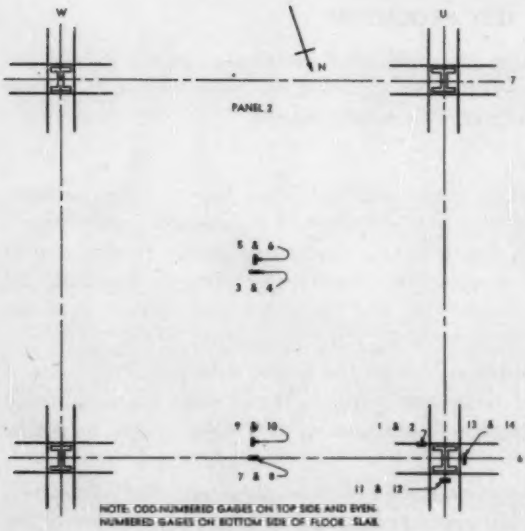


Fig. 5—Location of SR-4 strain gages

Location of strain gages

Fig. 5 shows the panel in which the strain gages were located as follows:

1. Top flange of steel channel close to column
2. Bottom flange of steel channel and opposite No. 1
3. Top of concrete slab; east-west direction
4. Underside of bottom reinforcing; east-west direction
5. Top of concrete slab; north-south direction
6. Underside of bottom reinforcing; north-south direction
7. Top of concrete slab; east-west direction
8. Underside of bottom reinforcing; east-west direction
9. Topside of top reinforcing; north-south direction
10. Underside of concrete slab; north-south direction
11. Topside of top reinforcing; east-west direction
12. Underside of concrete slab; east-west direction
13. Topside of top reinforcing; north-south direction
14. Underside of concrete slab; north-south direction

In addition to these gages, temperature compensating gages were used on blank specimens of concrete and encased steel, and a pair of 120-ohm precision resistors checked indicator drift.

Intensity and sequence of loading

To secure maximum bending moments in both column strip and middle strip, six panels were loaded as indicated by the crosshatching in Fig. 6. The full load intensity was 600 psf. Strain gage readings were recorded at one-quarter, one-half, and three-quarter loadings of Sequence A and after each of the loadings indicated by A, B and C in Fig. 6.

Auxiliary tests

In order to interpret correctly the results of the loading tests, stress-strain curves for both concrete and reinforced steel were obtained, as well as the ultimate strength values for both materials. In the case of the concrete, three SR-4 gages (type A-9) were attached 120 deg apart on the outside of 6 x 12-in. cylinders. The strains were recorded, averaged, and plotted for increments of 200 psi.

METHOD OF TEST

The area bounded by columns X-7, S-7, S-5, and X-5 was roped off and dial micrometers were placed in position at the center of each bay and as indicated in the sequence of loading diagrams, Fig. 6. SR-4 gages were placed as indicated in Fig. 5 and attached to the SR-4 strain apparatus.

The applied load consisted of 750 tons of salt (600 psf) bagged in 100-lb units loaded 32 bags to a skid. Sequence A was developed and deflection

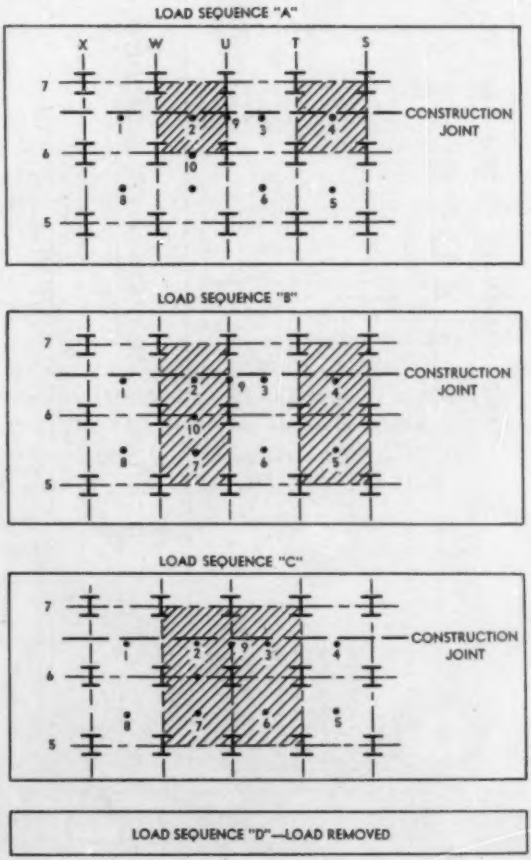


Fig. 6—The four loading sequences

TABLE 1—SUMMARY OF SR-4 GAGE READINGS

Gage		Unit stresses, kips per sq in. at loading sequence indicated										
No.	On	A/4	A/2	3A/4	A	A+24 hr	B	B+24 hr	C	C+24 hr	D	D+24 hr
1	Channel	+1.1	+3.4	+2.9	+4.4	+5.9	+7.5	+8.5	+9.8	+6.7	—	—
2	Channel	+0.3	+0.9	0.0	−1.1	+0.9	+1.6	+2.6	+2.1	+1.8	+5.3	+3.6
3	Concrete	0.00	−0.08	−0.16	−0.34	−0.34	−0.85	−0.88	−0.84	−0.17	0.00	0.00
4	Reinforcing steel	−0.1	+2.1	+2.4	+3.5	+6.4	+6.7	+8.1	+11.1	+4.5	−4.3	−5.3
5	Concrete	−0.07	−0.24	−0.30	−0.56	−0.55	−0.57	−0.49	−0.58	−0.41	−0.07	+0.04
6	Reinforcing steel	+1.5	+6.4	+6.4	+7.6	+8.5	+11.3	+12.2	+12.3	+8.0	−0.5	−0.7
7	Concrete	−0.02	−0.10	−0.11	−0.30	−0.33	−0.93	−0.90	−0.87	−0.98	−0.40	−0.26
8	Reinforcing steel	+0.6	+4.7	+5.0	+8.5	+9.5	+18.4	+18.9	+19.3	+16.6	+5.6	+4.5
9	Reinforcing steel	−1.4	+3.7	+1.1	+2.8	+5.2	+6.6	+7.0	+9.5	+9.6	+2.1	+2.8
10	Concrete	−0.01	+0.23	+0.06	+0.02	+0.01	−0.10	0.00	+0.12	+0.15	+0.30	+0.28
11	Reinforcing steel	+0.8	+2.4	+2.4	+2.7	+6.1	+3.8	+6.3	+11.7	+12.3	+0.1	−0.4
12	Concrete	0.0	−0.08	−0.11	−0.26	−0.20	−0.97	−0.85	−1.10	−0.77	−0.14	−0.39
13	Reinforcing steel	+0.6	+2.1	+1.9	+2.5	+4.2	+4.2	+5.6	+13.5	+16.9	+1.1	−1.2
14	Concrete	Gage readings useless; evidence of electrical interference										

TABLE 2—SUMMARY OF DIAL MICROMETER READINGS

Load sequence	Deflection, in.									
	Bay 1	Bay 2	Bay 3	Bay 4	Bay 5	Bay 6	Bay 7	Bay 8	Point 9	Point 10
A/4 (150psf)	−0.0050	+0.0560	0.0000	+0.0570	−0.0010	−0.0040	−0.0050	0.0000	+0.0150	+0.0200
A/2 (300psf)	−0.0060	+0.1045	−0.0020	+0.0970	−0.0050	−0.0040	−0.0050	0.0000	+0.0500	+0.0450
¾ A (450psf)	−0.0045	+0.1465	−0.0050	+0.1420	−0.0060	−0.0100	−0.0030	−0.0020	+0.0615	+0.0630
A (600psf)	−0.0100	+0.2235	+0.0050	+0.2295	−0.0060	−0.0150	−0.0020	−0.0050	+0.1140	+0.0950
A + 24 hr	−0.0100	+0.2640	+0.0220	+0.2700	−0.0070	−0.0150	−0.0020	−0.0050	+0.1320	+0.0990
B	−0.0230	+0.2915	0.0000	+0.2990	+0.2250	+0.0160	+0.1810	−0.0100	+0.1320	+0.1850
B + 24 hr	−0.0230	+0.2965	0.0000	+0.3020	+0.2300	+0.0200	+0.1880	−0.0150	+0.1300	+0.1990
C	−0.0230	+0.3055	+0.2500	+0.1200	+0.0420	+0.2350	+0.2100	−0.0150	+0.1600	+0.2000
C + 24 hr	−0.0250	+0.3115	+0.3100	+0.1050	+0.0150	+0.2800	+0.2230	−0.0150	+0.2000	+0.1950
D (Load removed)	−0.0200	+0.1245	+0.0530	+0.0320	+0.0080	+0.0840	+0.0450	−0.0050	+0.0800	+0.0220
D + 24 hr	−0.0020	+0.0380	+0.0380	+0.0250	+0.0080	+0.0180	+0.0380	+0.0050	+0.0420	+0.0150

Fig. 7—Full load of 600 psf applied to test area during Sequence C

readings were taken at 150, 300, 450, and 600 psf. The concrete slab under test was carefully inspected at each increment of loading for cracks or other signs of failure. The full load was allowed to remain in place for at least 24 hr, then placed in position for Sequence B. After 24 hr, Sequence C was developed, followed by a 24-hr recovery period. Fig. 7 shows the loading test in progress.

TEST RESULTS

The test results are summarized in Tables 1 and 2.

Reduction of the SR-4 strain readings to stresses was made as follows. Stresses for steel were obtained by multiplying micro-in. per in. as recorded by the SR-4 strain apparatus by the actual modulus of elasticity of 30,300 kips per sq in. as obtained in the control tests. Stress for concrete were taken from Fig. 8, stress-strain curve of the concrete cylinder control tests.

The results of the strength tests of the materials follow:

	Elastic limit, psi	Ultimate strength, psi	
		Average	Minimum
Concrete, 7-day test		3570	2900
Concrete, 28-day test		4900	4300
Reinforcing steel	46,100	79,700	77,600
Steel channels	41,550	63,500	61,500

Discussion of SR-4 gages

Attempts were made at precise control of the SR-4 readings by using two types of reference gages. The first consisted merely of two 120-ohm precision resistors connected to the SR-4 indicator at the end of each set of readings. A small drift of the indicator was noted and corrections were duly made to all gage readings. The second type consisted of temperature reference gages;

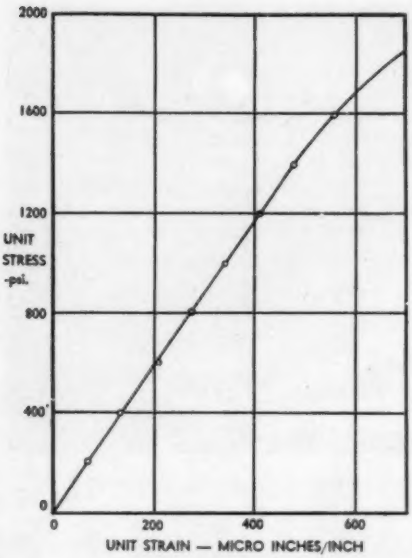


Fig. 8—Stress-strain curve for concrete control specimens

the idea was that these gages would undergo exactly the same changes of temperature as the gages attached to the key points on the test floor. These "dummy" gages presumably should change exactly as the test gages. Since these reference gages were on the circuit opposing the test gages, any temperature changes were theoretically balanced out.

The erratic behavior of Gage 14 was discovered after the loading was in progress, and it was considered hazardous to the operation of the other gages having wires in the same conduit to pull the wire leads for Gage 14 and replace them.

CONCLUSION

Visual examination of the test slab showed no signs of cracking or failure. Deflections and recovery were all within the special stipulations of the Philadelphia Building Department.

The stress-strain data taken by SR-4 gages indicate that none of the structural members under test reached the elastic limit.

These test results indicate that the "smooth ceiling" construction as used in this slab met the requirements of the Philadelphia Building Code, and official approval for its use in Philadelphia has been given.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

Heavy Media Processing of Gravels in New Brunswick*

By I. D. MacKENZIE†

SYNOPSIS

Materials of low specific gravity had to be removed from available local aggregate for the Beechwood development of the New Brunswick Electric Power Commission in order to produce concrete with satisfactory resistance to freezing and thawing. Heavy media separation plant used suspension of ferrosilicon and magnetite in water, removing aggregates which floated in the medium whose specific gravity was about 2.6. Aggregates which sank were cleaned and retained for concreting. Salvage and reuse of the suspension medium are explained.

INTRODUCTION

The Beechwood development of the New Brunswick Electric Power Commission is located on the St. John River approximately 100 miles upstream from Fredericton, N. B. The St. John, like the St. Lawrence, is an international and interprovincial river since its watershed extends into Maine and the province of Quebec. This development was constructed for the generation of power; initially there will be two units of 45,000 hp each, with provision for the installation of a third unit of similar size. Construction began in the early spring of 1955 and delivery of power began in November, 1957.

The major portion of the some 200,000 cu yd of concrete required for the development was in the nine-gate sluice section, the two-gate regulating section, and in the powerhouse and intake sections where the water passages are unlined. The river flows in a south-southeast direction at Beechwood so that much of the downstream face of the structures will be exposed to severe frost action, particularly during late winter. In the gate section the concrete must resist the erosive action of high velocity flood discharge as well as this frost damage.

AGGREGATE SOURCES

Field investigation to determine sources of concrete aggregates began in the spring of 1954. The bedrock at the site is an argillite with a well developed, in places almost a slate-like, cleavage obviously unsuited for concrete aggregate. As a result of Pleistocene glaciation and postglacial stream action, there are numerous gravel terraces and bars along and in the river starting

*Received by the Institute June 12, 1957; based on a paper presented at the tenth ACI regional meeting, Montreal, Quebec, Canada, Oct. 24, 1956. Title No. 55-7 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Member American Concrete Institute, Geologist, The Shawinigan Engineering Co., Ltd., Montreal, Quebec, Canada.

Fig. 1—Typical deterioration of structures over 30 years old; unprocessed local aggregates used in the concrete

above Beechwood and continuing many miles downstream. Some of the terraces rise over 50 ft above present river level and contain thousands of yards of material. A megascopic examination of the gravels in the faces of the numerous pits that had been opened up in these deposits near the site indicated that there was too high a percentage of unsound, porous, and nondurable particles for the material to be classed as a top quality concrete aggregate. The gravel particles consisted of soft and hard shales, porous and dense sandstones, mudstones, siltstones, lavas, igneous rocks, chunks of ochre, and crystalline quartz. Due to their origin, the makeup and local grading of these deposits was highly variable. An examination of some of the older concrete structures in the region (Fig. 1), reportedly made with these gravels, confirmed the opinion that concrete made with this material would not have the required durability.

The nearest known source of igneous rock was several miles away and to use this material it would be necessary to set up a quarrying operation and a crushing plant. After consultation, the New Brunswick Electric Power Commission authorized a complete investigation to determine the most economical source of concrete aggregates of the required quality. Sampling of all sources of aggregates and routine laboratory testing was begun. The local gravels met all the requirements including the soundness test until the results of freeze-thaw testing became available. Then and only then was the visual assessment confirmed. It was found that the sand as well as the gravel fraction was unacceptable.

UPGRADING AGGREGATE QUALITY

All efforts were then turned to seeking methods to upgrade the quality of the gravels. It was suspected that the troublemakers were the soft, porous shales, mudstones, siltstones, porous sandstones, and other particles in the gravels that had a low specific gravity. In 1948-49 at Rivers, Manitoba,

TABLE 1—COMPARATIVE TEST RESULTS ON CONCRETES MADE WITH GRAVITY-SEPARATED AGGREGATE FRACTIONS, 100 CYCLES OF FREEZING AND THAWING

Grading	Weight loss, percent	
	Concrete made with sink fraction aggregates, specific gravity greater than 2.60	Concrete made with float fraction aggregates, specific gravity less than 2.60
No. 4 to ⅜ in.	0	9.8
⅜ in. to ¾ in.	4.82	21.2
¾ in. to 1½ in.	1.29	10.7
1½ in. to 2½ in.	0	10.9

the Royal Canadian Air Force had successfully and economically improved a local gravel by the removal of the low specific gravity particles using equipment developed by the mineral industry.* This process is known as the heavy media or sink-float process and is capable of making a close separation on material retained on the No. 10 sieve.

A trial separation was made in the laboratory of the Federal Department of Mines and Technical Surveys in Ottawa where equipment was available. The gravel particles were separated at various specific gravities and it was found that the particles remaining after the lighter 15 to 20 percent of the material had been removed were resistant to damage when subjected to freeze-thaw testing. A larger sample was separated, trial air-entrained mixes were made, cured, and tested in the freeze-thaw unit. Sand of known quality was used in these mixes. The concrete with the heavy aggregate had the required resistance; that made with the light fraction had very poor resistance (Table 1).

It was decided that the gravel—that is, the material larger than ⅜ in.—would be processed and that the sand fraction would be discarded due to difficulties in carrying out gravity separations on fine materials. An excellent concrete sand available from St. John Harbor could be brought in by rail.

Heavy media separation plant

In order to make the necessary gravel separation, a drum-type heavy media plant was purchased. This unit came preassembled to the extent rail shipment would allow so that erection time was minimized. The unit required one operator, the connected power load was 75 hp, and it required approximately 500 U. S. gal. per min of water. This unit was known at the job as the sink-float plant.

The separation of materials by specific gravity is based on the introduction of the particles of varying specific gravity to a suspension or medium maintained at the specific gravity at which the separation is to take place. Particles that are lighter float like a piece of wood on water and the heavier particles sink.

*Trites, C. V., and Shannon, J. D., "Acceptable Aggregates from Low-Grade Deposits," *Rock Products*, V. 53, No. 2, Feb. 1950.

†Trites, C. V., and Shannon, J. D., "Mining Process Applied to Runway Construction," *The Engineering Journal* (Canada), Apr. 1949.

Water has a specific gravity of 1.0 and a Canadian gallon of water weighs 10 lb. If 16.0 lb of a heavy powder such as magnetite or ferrosilicon is added to a gallon of water, the volume of the mixture is increased but slightly whereas the weight has increased to 26.0 lb, so that, if it is agitated to keep the powder in suspension, the effective specific gravity of the suspension or medium is nearly 2.60. Particles heavier than this will sink and the lighter will float.

The sink-float plant is a series of pumps, piping, and other equipment which maintain at a definite specific gravity a suspension or medium to which the particles to be separated are introduced.

Processing the aggregate

The aggregate processing plant set up at the Beechwood gravel pit (Fig. 2) handled material which was excavated with a ¾ cu yd shovel, hauled by truck to a charging hopper, and thence by a conveyor to a dry vibratory screen where most of the sand was removed. The oversize from this screen went to a twin jaw crusher; the crusher product and the undersize gravel were recombined and were passed over a second dry screen where further sand was removed. The gravel was then passed over a horizontal washing screen to remove adherent fines. For successful operation of a gravity separation it is essential that all possible fines be removed.

The washed gravel was then fed into the drum of the sink-float plant (Fig. 3). This drum, 8 ft long and 8 ft in diameter, was rotated at approximately 2 rpm, and was kept one-third full of the medium which consisted of a suspension of ferrosilicon and magnetite in water; the suspension was constantly being pumped into the drum and flowed out through the discharge chutes. The lighter float particles were carried along on top of the flowing medium to the float discharge chute and thence to the float section of the washing screen. The sink material went to the bottom of the drum and was picked up by lifters attached to the inner circumference of the drum; the lifters dropped it onto a second chute which discharged onto the sink portion of the vibrating washing screen. This washing screen was divided longitudinally with approximately 25 percent of the screen surface reserved for the float material and 75 percent reserved for the sink material. On the

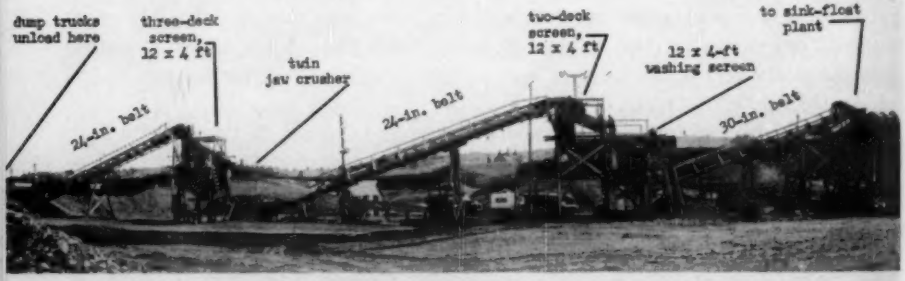


Fig. 2—Aggregate processing plant. Pit run material entering at left, gravity separated coarse aggregate discharged from sink float plant on extreme right

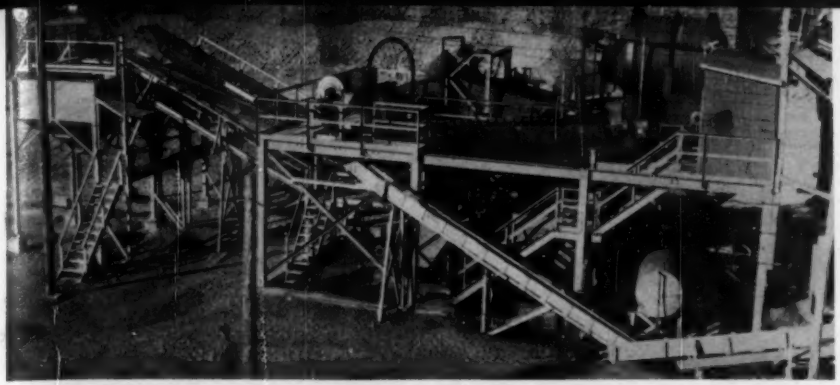


Fig. 3—Sink-float plant. Washed coarse aggregate enters drum on right; gravity separated aggregate discharged from belts to separate bins on left

drum end of this screen the water, the ferrosilicon, and the magnetite began to drain off. Further along, high pressure water jets washed the medium from the gravel. The sink and float material (Fig. 4 and 5) were conveyed to surge bins, the float being trucked to waste and the sink fraction being trucked to the job site where it was separated into three sizes and stockpiled.

Medium salvaged, reused

The suspension or medium that was washed from the aggregates was pumped to a magnetic separator where the excess water and contaminants were discarded. The clean medium was then pumped to a spiral classifier where excess water overflowed to waste and the medium moved to the head of the unit where it was passed through a demagnetizer and thence back into the circuit. The spiral or screw on this classifier is adjustable so that the amount of settled medium remaining in the classifier can be increased or decreased. In this way the amount of solids in the suspension can be controlled and the resultant specific gravity of the suspension or medium can be varied as required.

In the operation of such a plant it is important to maintain the density of the medium. Determinations of medium density are simple and quick, and changes in density can be easily made by varying the clearance on the spiral in the classifier.

To keep the cost of the separation down, it is essential that the loss of medium be kept to a minimum. This can only be done by a thorough washing of both the sink and the float fractions. Every pound of media going out of the plant represents so many more cents per ton on the cost of the processing.

At Beechwood this plant operated from the late summer of 1955 until freeze-up and from early spring of 1956 into August for a total of 330 10-hr shifts. The production of finished aggregate was some 260,000 tons—nearly 80 tons per hr or slightly more than 6 percent greater than the rated capacity of the machine. For short periods production was appreciably higher.

To maintain the specific gravity of the medium, both ferrosilicon and magnetite were used. The over-all consumption was 0.543 lb of ferrosilicon and 1.12 lb of magnetite per ton of finished gravel. A major portion of the

Fig. 4—Sink fraction of coarse aggregate. Particles are smooth, heavy, and hard. Colors generally dark greys, browns, black

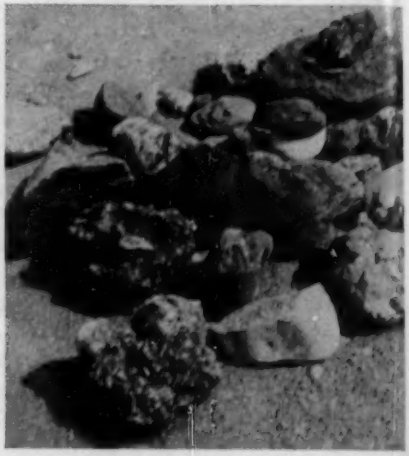


Fig. 5—Float fraction of coarse aggregate. Particles are light, soft, and frequently have rough surfaces. Color, generally light shades of brown and grey

gravel particles were partially flattened and most of the medium lost adhered to the underside of these particles. Upward-directed spray nozzles placed above and below the intermediate deck of the screen would have washed the underside of these particles and would have substantially reduced the loss of medium.

CONCLUSIONS

As a result of the heavy media processing, coarse aggregate for the Beechwood development was obtained for less than half the estimated cost of production from any other source. The process, if normal supervision is provided, is relatively trouble-free. It could be advantageously investigated by many producers of coarse aggregate who are having difficulty meeting the durability clause in specifications.

ACKNOWLEDGMENTS

The laboratory sink-float testing was done at the Industrial Minerals Division of the Department of Mines and Technical Service, Ottawa. M. F. Goudge is director of this department, and the testing was supervised by Victor A. Haw. The preliminary aggregate investigation and routine laboratory testing, including freeze-thaw testing, were done by Associated Designers and Inspectors, Fredericton, N. B., and Warnock Hersey Co., Montreal.

J. T. Donald and Co. supplied job inspection, and Terminal Construction Co. operated the gravel plant and furnished the production data. Foundation Maritime was the general contractor for the development, and the designers and consultants for the project were The Shawinigan Engineering Co., Ltd. All these firms are in Montreal.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

Load Test of 120-ft Precast, Prestressed Bridge Girder*

By FAZLUR R. KHAN† and ANDREW J. BROWN‡

SYNOPSIS

General design data are given for four prestressed concrete highway bridges and two prestressed concrete railroad bridges, built at the U. S. Air Force Academy site. One full size 120-ft girder was tested in two stages, leading to final destruction, in order to check workmanship and validity of design assumptions. First the girder alone was loaded to cracking. Then a 9 ft wide and 7 in. thick slab was cast to form a composite unit which was loaded to failure. Results of both tests are discussed in detail.

GENERAL DESCRIPTION

Four four-lane prestressed concrete highway bridges ranging from 240 to 600 ft long and two prestressed concrete railroad bridges, each 144 ft long, were planned for U. S. Air Force Academy in Colorado Springs as a part of the site development.

All the highway bridges have simply supported spans of 120 ft. The deck construction consists of precast, prestressed concrete girders spaced 9 ft on centers. The diaphragms are spaced at 20 ft. The 7-in. deck slab is cast in place. The highway bridges are designed for H20-S16-44 AASHO loading. The modified T-girder is 71 in. high; the 26-in. base tapers to 8 in. at the neck and flares out to 36 in. at the top flange (Fig. 1).

The girder is designed for a concrete strength of 5500 psi at 28 days and 4500 psi at time of prestressing. Concrete design stresses used are 2475 psi compression and 200 psi tension at transfer of prestress, and 2200 psi compression and 136 psi tension at working loads.

In the final condition a 9 ft wide, 7 in. thick slab of 3000-psi concrete acts with the precast girder as a composite unit. The composite section was designed for a live load moment of 2010 ft-kips, including impact; dead load moment, including precast girder, cast-in-place slab and 2-in. bituminous wearing surface, is 4027 ft-kips. Total working load moment thus becomes 6037 ft-kips.

Each girder is post-tensioned by the Prescon method with 12 cables each having 16 wires of ¼-in. diameter. Prestressing wire has tensile strength of 245,000 psi and minimum guaranteed elongation of 4 percent at rupture. It

*Received by the Institute Sept. 27, 1957. Title No. 55-8 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each. Discussion (copies in triplicate) should reach the Institute not later than Oct. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Member American Concrete Institute, Civil Engineer, Dacca, Pakistan; formerly Senior Structural Designer, Skidmore, Owings, and Merrill.

‡Chief Structural Engineer, Skidmore, Owings, and Merrill, Chicago, Ill.

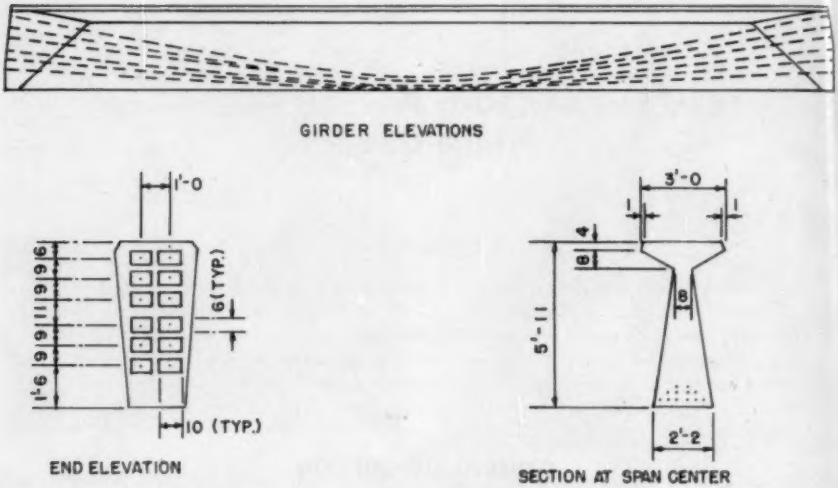


Fig. 1—Girder details

has a Young's modulus of 29.1×10^6 psi. Wire stress at transfer was 176,000 psi.

Two skew bridges for the railroad are designed for Cooper E-60 loading. The precast, prestressed girder dimensions are the same as for the highway bridges, but lower design stresses are used and no tension is allowed at working loads.

From an economical point of view, the efficient section is one with minimum web area. This generally leads to a T-section or I-section. However, esthetic requirements controlled the final choice of section and a small amount of economy was sacrificed.

The section with the tapered bottom has one definite advantage. Due to the 26 in. wide base and gradual taper, it is considerably simpler to arrange the prestressing tendons throughout the length of the girder. The 8-in. neck allows pairs of tendons to pass through without impairing spacing or clearance requirements. Also, due to the wide base it was possible to arrange the tendons with their center of gravity only 6 in. from the bottom of the girder. This is seldom possible with ordinary flanged sections.

PREDICTED ULTIMATE MOMENT

For computing the ultimate moment capacity of an underreinforced beam only two quantities have to be known. One is the lever arm at failure and the other is the steel stress at failure. To compute these two quantities in a beam, several theories have been developed based on experimental data. These theories all rest on the assumption that at failure the section above the neutral axis is rectangular. For almost all commonly used sections, the above

assumption is true since the neutral axis at ultimate loads moves up into the flange area.

A statistical study had been made from 35 beam test results reported since 1940.[1] Test results were analyzed for determining the steel stress $\alpha f_s'$ and the lever arm jd at failure. The values of α and j were plotted against a dimensionless parameter $Q' = E_s p/Kf_c'$ where E_s is the elastic modulus for steel, p is the effective percentage of steel, and Kf_c' is the average compressive stress of concrete at failure. f_c' and f_s' are ultimate strength of concrete and steel respectively. Values of K were computed according to Billet and Appleton,[2] where

$$K = \frac{3000 + \frac{1}{2} f_c'}{1500 + f_c'}$$

for $f_c' = 3000$, $K = 1$.

Fig. 2 and 3 indicate that both α and j vary nearly linear with Q'. Lines drawn will have the following equations for α and j.

$$\alpha = 1 - 0.00258\, Q'$$

$$j = 1 - 0.00325\, Q'$$

On the basis of these equations, the ultimate moment for an underreinforced beam is expressed as

$$M_u = A_s \alpha f_s' jd$$

$$= \gamma A_s f_s' d$$

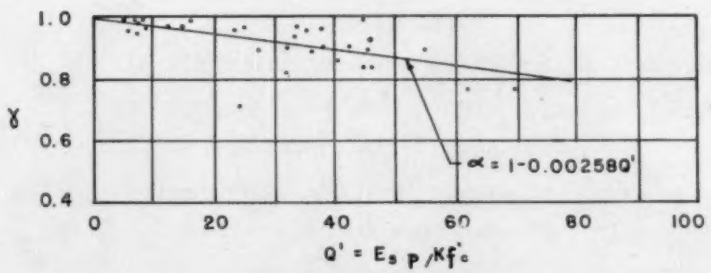


Fig. 2—Diagram of α values

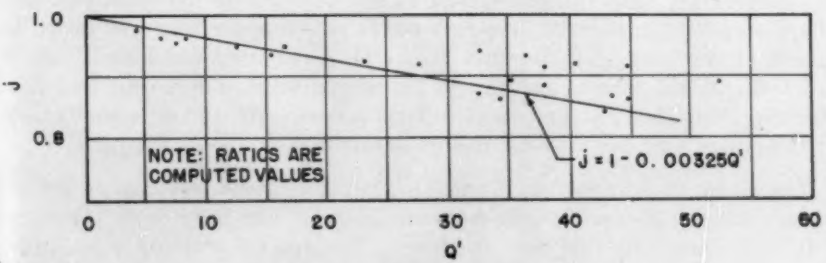


Fig. 3—Diagram of j values

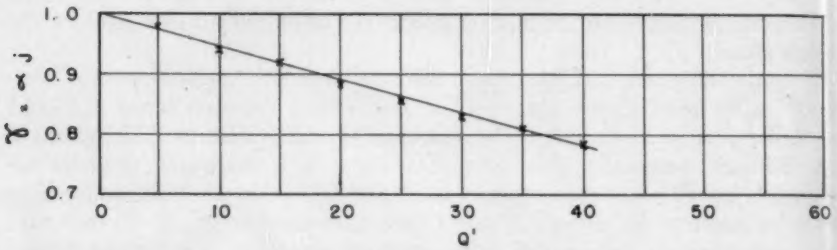


Fig. 4—Diagram of γ values

The factor $\gamma = \alpha j$ is computed and plotted for corresponding values of Q' in Fig. 4.

For the 120-ft girder

$$Q' = 29.1 \times 0.00125 \times 10^6/3000 = 12.2$$

for which $\gamma = 0.93$. Therefore, the predicted ultimate moment for cables is

$$M_u' = 0.93 \times 9.384 \times 245 \times 69/12$$

$$= 12{,}350 \text{ ft-kips}$$

Ultimate moment capacity for mild steel* can be computed with reasonable accuracy by taking the same value of γ

$$M_u'' = 0.93 \times 1.76 \times 32 \times 69/12$$

$$= 300 \text{ ft-kips}$$

Therefore, total $M_u = 12{,}350 + 300 = 12{,}650$ ft-kips.

Predicted jack load at failure is given by the equation

$$12{,}650 = 145\,P + (4027 - 392)$$

392 ft-kips is dead load moment due to 2-in. wearing surface.

$$P = 9015/145 = 62.5 \text{ kips} = 31.25 \text{ tons}$$

TEST LOAD ARRANGEMENT

To check the basic design assumption and to evaluate the effectiveness of the composite construction, the first 120-ft prestressed girder was tested to failure in two stages. In the first stage, the girder itself was loaded until it cracked and was then unloaded. In the second stage, a 9 ft wide and 7 in. thick concrete slab was cast on it to form a composite unit as actually used in the bridge, and this unit was loaded to failure.

The girder was supported on two 3 x 3-ft piers. Load was applied by ten equally spaced 50-ton capacity hydraulic jacks. The jacks were placed on 3 ft wide beams to distribute the load. The upward reaction was carried

*Yield point of mild steel is 50-60 percent of ultimate and occurs much before yield point of high strength cables (which is 80-90 percent of ultimate) is reached.

by 10-ft cross beams which were anchored into subpiers. The subpiers, 2 ft in diameter, with 3 ft diameter bells were drilled in, to a depth of 15 ft, to resist the pull from the tie-down cables.

INSTRUMENTATION

Electrical strain gages

SR-4 gages, Type A-9 with 6-in. gage length, were used to measure strains at the top and bottom surface of the girder. These gages were applied at midspan and at the quarter points. Three gages were applied at each face in each cross section to reduce possibility of erratic readings and also to detect lateral movement of the girder, if any. Strains were read with a Baldwin portable strain indicator. Dummy gages for temperature compensation were mounted on unstressed concrete cylinders.

Mechanical strain gages

The distribution of strain in the concrete over the depth of the beam was measured with 10-in. Whittemore strain gage, reading to the closest 0.0001 in. Strains at the two quarter points and the midpoint were measured on both sides of the girder. The gage lines were at 6 in., 1 ft 6 in., 4 ft 6 in., and 5 ft 6 in. below the top of the girder (Fig. 5). Steel plugs ½ in. in diameter and ¼ in. long with gage holes were cemented to the sides of the beam to form the gage lines.

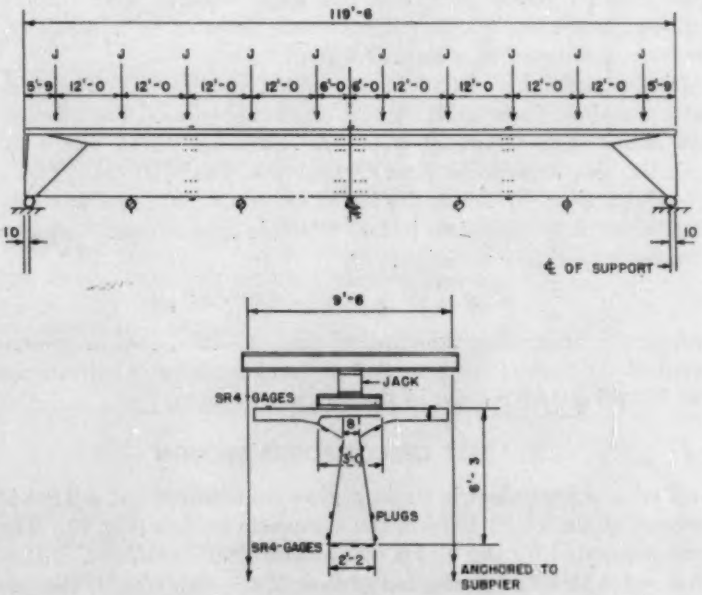


Fig. 5—Test load arrangement and instrumentation

Fig. 6—Load test in progress on composite section

Dial indicators

Deflections were measured at midspan and at every sixth point with dial gages reading to 0.001 in. For simultaneous check, steel scales reading to 0.01 in. were attached to the side of the girder and readings were recorded by transit.

Dial as well as transit readings were taken also at the end supports, and girder deflections were corrected for end support settlement.

TEST OF PRECAST GIRDER

The girder was loaded in 2.5 ton increments per jack. After every increment the electric, Whittemore, and dial gage readings were recorded. At about 15-ton jack load, fine hair cracks were detected at midspan of girder. The test was completed at a load 17.5 tons.

A maximum deflection of 3.65 in. was noted at midspan. The load-deflection curve remained linear until the first cracks appeared. The average elastic modulus of concrete computed from the deflection curve was $4.63 \times 10.^6$ The cylinder test strength at time of load test was 5700 psi. The average elastic modulus given by the strains in the electric strain gages was 4.31×10^6 psi. The average of these two is 4.47 which is approximately equal to that given by the emperical expression:

$$E_c = 1.8 \times 10^6 + 460 f_c' = 4.42 \times 10^6 \text{ psi}$$

Based on the reasonable assumption that the total loss in prestress was 10 percent at the time of the test, the apparent modulus of rupture was computed as 833 psi or 14.6 percent of the concrete strength.

TEST OF COMPOSITE SECTION

After Test 1, the top electric strain gages were removed and a 9 ft wide 7-in. slab was cast on the girder to form the composite section (Fig. 6). The formwork was supported by the girder to simulate field conditions. SR-4 gages, Type A-9, were attached to the top surface of the slab after it was hard and dry, and all other electric and mechanical gages were left in position.

Loads were applied in 2.5-ton increments per jack. Readings of all the gages and scales were taken at every increment of load until a jack load of 22.5 tons was reached. Load of 22.5 tons per jack was maintained 24 hr, and deflection readings were taken every 30 min. This load corresponds to about 75 percent of the predicted ultimate capacity. After 24 hr the girder was unloaded and recovery measured. The additional recovery 35 min later was very small.

The girder was then reloaded and all gages were read at load increments of 7.5 tons up to 22.5 tons. Beyond 22.5 tons all gages were read at 2.5-ton increments. Most of the SR-4 gage readings were erratic above the 22.5-ton load. Beyond 27.5 tons the rate of deflection became very rapid and only the dial gages, the steel scales, and the Whittemore gages were read. When the jack load was approximately 30 tons, the beam yielded. Further jacking resulted in increased deflection, while indicating considerable reduction in jack pressure. At yield all cracks near midspan had opened up all the way beyond the neck and the deflection was about 15 in. Since further jacking would have required time-delaying shimming operation, it was decided to stop jacking. From the behavior of the girder it is apparent that the collapse load would not be appreciably greater than the yield load.

TEST RESULTS

Cracking pattern

Cracks in the composite section were observed first near midspan at 12 tons load. Since the beam was already cracked in Test 1, cracks opened up soon after bottom stress reached zero. As the load was increased, more cracks appeared. Fig. 7 shows the crack pattern at a load of 22.5 tons. The cracked area as seen is approximately a parabola. All cracks were practically vertical and were spaced uniformly about 12 in. apart. In post-tensioned beams where the cables are not grouted there is a tendency toward development of fewer but wider cracks. Proper grouting develops bond between the prestressing steel and concrete and results in a larger number of closely spaced cracks. It is evident from the test girder crack pattern that grouting was effective.

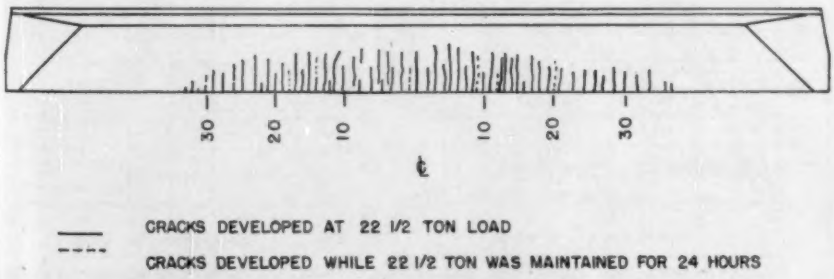


Fig. 7—Crack pattern

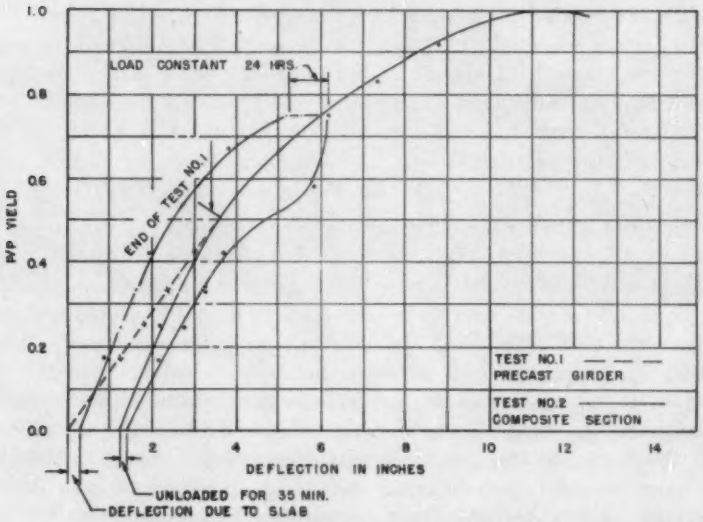


Fig. 8—Load-deflection curve for midspan

Although the neutral axis at load near the ultimate moves up close to the 8-in. neck, inclined diagonal tension cracks did not develop.

Deflection

Fig. 8, 9, and 10 show the complete load-deflection diagrams for the midspan, the third points, and sixth points. The curves for Test 1 give a fair comparison of the relative stiffness of the girder itself with the composite section's stiffness.

The load-deflection curves for the three points measured are similar in shape. The curve for the midspan deflection is linear up to about 12 tons

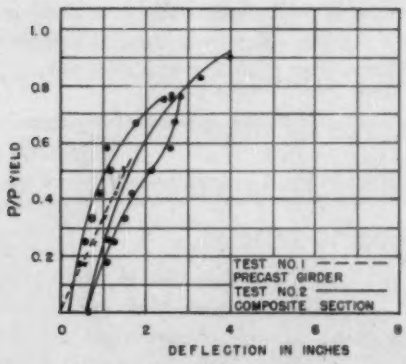


Fig. 9—Load-deflection curve for one-sixth point

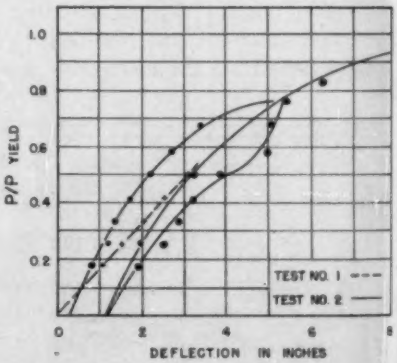


Fig. 10—Load-deflection curve for third point

jack load. Beyond this, cracks near midspan opened up, causing an increased rate of deflection. At 75 percent of the yield load (22.5 tons), the total deflection was 5.55 in. During the 24 hr this load was held, the girder deflected another 0.6 in. The time-deflection curves over 24 hr and over the first 4 hr are shown in Fig. 11 and 11a.

As the load was gradually released, up to about 58 percent of the yield load there was very little recovery of deflection. This is mostly due to the phenomenon of plastic lag. However, the rate of recovery soon increased. From the last part of the release curve it appears that the elastic modulus of concrete at loading is greater than at unloading. Average elastic modulus for uncracked section at loading is computed as 4.6×10^6 psi, and the average elastic modulus for same loads computed from the release curve is $3.28 \times 10.^6$ The reduced apparent elastic modulus of concrete is a compensating aftereffect of the earlier plastic lag.

Depth of neutral axis

The main object of using the Whittemore gages was to find the position of neutral axis at different loads and to detect lateral movement of the girder during test. The gages on both sides of the girder read practically the same until the end of the test, proving that there was no significant lateral move-

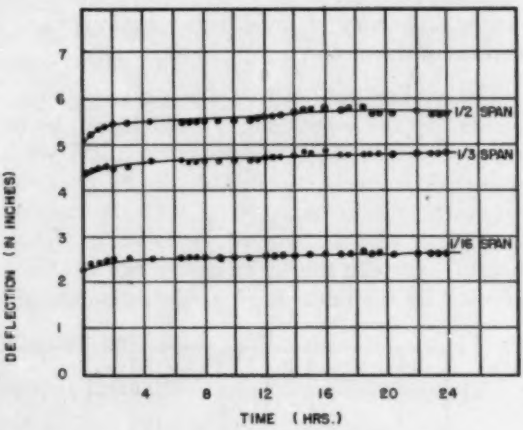


Fig. 11 — Time-deflection curve under sustained load equal to 75 percent of yield load

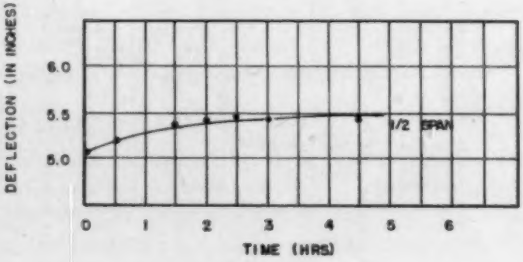


Fig. 11a—Enlarged part of Fig. 11

ment of the girder. The position of the neutral axis at every increment of load was computed for the midspan and the two quarter points as shown in Fig. 12 and 13. Up to the cracking load the measured position of neutral axis coincided with the computed theoretical location. After cracking, the neutral axis began to shift toward the top flange. At midspan the rate of rise of neutral axis was slower at lower loads. However, at about 70 percent of the ultimate load the neutral axis took a jump and from then on, till yielding, the rate of rise was practically linear. At 92.5 percent of ultimate moment the ratio of depth of neutral axis to the effective depth became 0.09. At this load the neutral axis is well within the flange area.

For the quarter point the ratio of depth of neutral axis to the effective depth is plotted against the ratio of actual total moment M_t to the computed ultimate moment $M_u = 0.93\, A_s f_s' d$. Just after the cracks opened, the neutral axis rose rapidly. This rate slowed down at about 0.6 of the ultimate moment and then rose at a fairly linear rate.

The behavior of the neutral axis at the midspan was different from that at the quarter point only at loads just after cracking. At midspan the rate of rise at this stage was slower probably due to the action of the mild steel reinforcement which tends to lower the neutral axis up to yield stress. At quarter point, the sudden jump of the neutral axis was due to the fact that the center of gravity of prestressing reinforcement is 13.5 in. above the bottom surface of the girder.

The average rate of rise of neutral axis both at midspan and at the quarter points was the same. Fig. 12 and 13 can be considered to be drawn for two separate beams, both underreinforced and yet having different percentage of prestressing steel, effective depth of steel, and sectional properties after cracking. Since the slopes of both the curves are the same, it leads to the conclusion that as long as the beam is underreinforced, the rate of rise of neutral axis with respect to percentage of ultimate moment is not appreciably affected by the percentage of reinforcement or by the proportions of the sec-

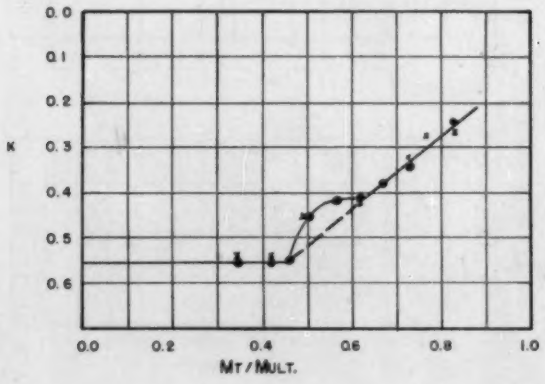


Fig. 12—Location of neutral axis at quarter point

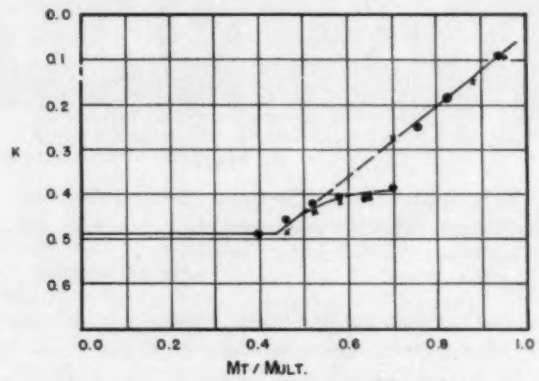


Fig. 13—Location of neutral axis at midspan

tion. It is felt that further research in this direction is required before a more general statement can be made.

Factor of safety against cracking

Moment at which the cracks opened in Test 2 is not the true cracking moment because the girder was already cracked in Test 1. From the first test an apparent modulus of rupture of 833 psi was computed. This can be also assumed for Test 2. Section modulus for bottom fiber of composite section is 26,800 in.^{3} and assuming all losses to have taken place, the superimposed moment that would cause cracks would be

$$M = (1031 + 833) \times 26{,}800/12$$

$$= 4180 \text{ ft-kips}$$

Therefore, total cracking moment

$$M_c = 4180 + (4027 - 392) = 4180 + 3635$$

$$= 7815 \text{ ft-kips}$$

The total load factor of safety against cracking is

$$F_{tc} = 7815/6037 = 1.3$$

and live load factor of safety against cracking is

$$F_{lc} = (7815 - 4027)/2010 = 1.88$$

Most of the prevalent specifications, except the Swiss, do not require any factor of safety against cracking. However a higher live load factor of safety against cracking reduces the possibility of deterioration due to weather.

Factor of safety against ultimate loads

Ultimate moment was predicted to be caused by 31.5 tons per jack. In test, the girder failed at a jack pressure of 30 tons. With the additional dead weight of the distributing beams and the men, the actual load per jack was well above 30 tons, approaching the predicted failure load within a few percent.

The total load factor of safety against failure is, therefore,

$$F_{tu} = 12{,}650/6037 = 2.1$$

and live load factor of safety against failure is

$$F_{lu} = (12{,}650 - 4027)/2010 = 4.3$$

It is apparent that the design was controlled by the factor of safety against total load. The live load safety factor on such spans seldom controls design.

CONCLUSION

1. Development of vertical cracks only indicates that the modified T-shape is adequate for ultimate shear requirements although it has a narrow neck.

2. Proper grouting of cables is justified. Equal distribution of a large number of cracks indicates adequate bonding of prestressing steel. Test load proves that ultimate moment capacity computed for fully bonded condition is developed by efficient grouting.

3. Load-deflection curve for the release of high sustained load shows the effect of "plastic lag" at the beginning of release. Later rate of recovery indicates apparent modulus of elasticity of concrete lower than that at start of test.

4. The rate of rise of neutral axis in an underreinforced beam is practically proportional to the rate of increase of total moment. For two underreinforced sections having different proportions and percentage of reinforcement, the ratio of depth of neutral axis to the effective depth is approximately same for equal percentage of ultimate moment applied on each section.

ACKNOWLEDGMENTS

Construction is under the direction of the Air Force Academy Construction Agency, headed by Col. A. E. Stoltz as director. C. L. Tyler and E. A. Merrill head the Skidmore, Owings, and Merrill Colorado Springs office.

A. S. Horner of Denver has the general contract. A careful job of applying all gages and taking the readings was done by Leonard J. Mitchell, materials engineer, on leave from Bureau of Reclamation.

REFERENCES

1. Khan, F. R., and Appleton, J. H., "A Study of Tests on Prestressed Concrete Beams," Part B, Third Progress Report on the Investigation of Prestressed Concrete for Highway Bridges, Engineering Experiment Station, University of Illinois, June 1954.

2. Billet, D. F., and Appleton, J. H., "Flexural Strength of Prestressed Concrete Beams," ACI JOURNAL, June 1954, *Proc.* V. 50, pp. 837-854.

3. Siess, C. P., "Strength of Prestressed Concrete Members," *Paper* No. 3, Session B, Symposium on the Strength of Concrete Structures, London, 1956.

Discussion of this report should reach ACI headquarters in triplicate by Oct. 1, 1958, for publication in the March 1959 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Restraining conditions of large bridge slabs in reinforced concrete supported by heavy edge girders resting on columns (in Slavonian)

S. LAPAJNE, *Gradbeni Vestnik* (Ljubljana), V. 8, No 47-50, 1956-57, pp. 123-127

Reviewed by J. J. POLIVKA

Thorough investigation and structural analysis of such flat slabs, demonstrated on example of an 11 in. thick slab spanning 23 ft between two 24 x 30-in. edge girders which are supported by 24 x 24-in. columns 19 ft high. Special attention is given to lateral forces.

Simplified analysis of bridge arches rigidly connected with the upper structure (in Slavonian)

SVETKO LAPAJNE, *Gradbeni Vestnik* (Ljubljana), No. 45-46, 1956-57, pp. 87-91

Reviewed by J. J. POLIVKA

Thorough study was made in connection with the Lesce-Bled bridge across the Sava River. Fundamental methods of distributing bending moments in the arch, and the spandrel structure in accordance with the stiffness of individual members are given. Influence lines for vertical loads are developed for full and partial restraint of the arches. It is first assumed that the moments of inertia of the arch and the spandrel beam follow equal variation, and additional corrections are introduced.

Bridges and their builders

DAVID B. STEINMAN and SARAH RUTH WATSON, Dover Publications, Inc., New York, Revised Edition, 1957, 408 pp., $1.95

A narrative of adventure and dramatic interest which sets the story of each bridge against a background of the life of the times in which it was built, this should be enjoyably constructive reading for both engineer and layman. Outstanding bridge structures are described in historial periods ranging from primitive times, through the Roman era, Middle Ages, Renaissance, etc., through the twentieth century. Section on "man-made masonry" traces use of concrete as a bridge material from a 20-ft arch span built in 1889 through contemporary applications of prestressed concrete, describes the rigid frame concrete bridge and floating concrete spans.

Construction

Critical analysis of traditional and modern building methods (in Dutch)

B. H. H. ZWEERS, *Cement* (Amsterdam), V. 9, No. 5-6, June 1957, pp. 185-186

Reviewed by JOHN W. T. VAN ERP

Article compares building systems with a completely mechanized and continuous production line and others with combined cycles of operation. Examples are given of traditional building methods and typical defaults, as far as efficiency goes, are indicated. Concludes that for maximum productivity standard units as large as can be handled, must be prefabricated complete with all accessories and finish.

Unusual shell structure at Brussels World's Fair (in Dutch)

H. C. DUYSTER, *Cement* (Amsterdam), V. 9: No. 9-10, Oct. 1957, pp. 384-385; No. 11-12, Dec. 1957, pp. 447-450

Reviewed by JOHN W. T. VAN ERP

A building constructed entirely out of hyperbolic paraboloid shells was built for the Philips Co. of Holland, a large electronics firm. The design by Le Corbusier called for

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 1, July 1958, *Proceedings* V. 55. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

thin shells. These shells of 2-in. thickness were built up out of 4 x 4-ft prefabricated units. These 2000 units, all of them curved and skew, were cast on a sandbed stabilized with cement grout.

After erection prestressing was by wires on both sides. A preliminary stress analysis had been done on a scale model, and also a complete plywood model was built and prestressed as a check on the data as found from the other model test. All concrete being kept under compression, the only finish was a waterproof paint directly on the concrete. No other roofing material was required. Edges of shells were formed by ribs of cylindrical cross section, cast in place and also prestressed to take care of bending as well as torsional stresses.

Drinking water storage tank of 38,000 cu yd capacity at Rotterdam (in Dutch)

J. Aarnoudse, *Cement* (Amsterdam), V. 9, No. 7-8, Aug. 1957, pp. 301-308

Reviewed by John W. T. Van Erp

To cope with peak hour demands considerable storage capacity had to be provided. Tank is a concrete box about 200 x 230 x 22 ft high resting on 600 concrete piles. Roof is flat slab structure on square columns about 20 ft on centers; floor and walls are 20 in., roof, 16 in. thick. For minimum shrinkage cement similar to ASTM Type I was used with lowest possible W/C ratio and revibration. Concrete was kept wet until tank was filled.

Haniel high-rise garage in Düsseldorf (Die Haniel-Hochgarage in Düsseldorf)

Wilhelm Goertz, *Der Bauingenieur* (Berlin), V. 32, No. 4, Apr. 1957, pp. 126-130

Reviewed by Aron L. Mirsky

Detailed description of construction of four-story garage, with ramps on the outside of the structure, suspended from cantilevered roof girders. Associated with garage is a two-level structure of interesting concept: the ground floor, almost entirely unenclosed, is occupied by a gasoline station and a cashier's office so arranged that it supervises the entire facility; second story, supported by T-shaped concrete elements, is given over to a restaurant and small hotel. (For brief description of these structures in English, see *The Engineer*, V. 197, No. 5119, Mar. 5, 1954, pp. 363-364; "Current Reviews," ACI Journal, Oct. 1954, *Proc.* V. 51, p. 213.)

Concrete structures for the railroads in and around Rotterdam (in Dutch)

S. Noyon, *Cement* (Amsterdam), V. 9, No. 3-4, Apr. 1957, pp. 91-104

Reviewed by John W. T. Van Erp

War damage has brought about extensive changes in the rebuilding of the nucleus of Rotterdam, including an almost entirely rebuilt railroad network with new station buildings. Most of the many underpasses required have been built as tunnels of complete concrete box sections. This proved to be most economical foundation-wise. Other interesting structures are the extensive shell roofs over the station platforms—graceful structures of "butterfly" cross section (3 in. thick shells).

Construction Techniques

Plastering and rendering. Part 1

V. Saretok, *Bulletin*, International Association of Testing and Research Laboratories for Material and Structures (RILEM) (Paris), No. 38, 1957, pp. 5-89

Presents a very comprehensive report (translated from Swedish) on the historical development, materials, performance, and methods of application of plaster and stucco. Part 1 includes the entire bibliography of 218 selected references.

Repairs to concrete and masonry structures

C. K. Chokshi, *The Indian Concrete Journal* (Bombay), V. 32, No. 1, Jan. 1958, pp. 28-30

Describes methods of repair and strengthening existing concrete and masonry structures. Methods described include pressure grouting, concrete replacement, pneumatic application of mortar, and prepacked concrete.

Dams

Concrete dams (Beton-Staumauern)

Friedrich Tolke, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 15, May 21, 1957, p. 686

Reviewed by Aron L. Mirsky

Annual review (see also "Current Reviews," ACI Journal, Mar. 1957, *Proc.* V. 53, p. 908), summarizing recent developments in concrete dam design and construction.

Earth and stone dams are covered in other reviews in the same issue.

Register of dams in the United States

T. W. Mermel, McGraw Hill Book Co., New York, 1958, 444 pp., $12.50

Provides the essential statistics on over 2800 important dams in the United States. Gives name, location, structural data, ownership, engineer, and contractor. Lists dams alphabetically by name with a cross reference of reservoir names. Includes pictures of over 300 dams. Also presents a summary of state laws dealing with dams.

Design

Addenda—Cylindrical thin shell roofs of circular section (in French)

N. Dehousse, *Bulletin*, CERES (Liège), V. 7, 1955, pp. 3-118

Original review of this article appeared in "Current Reviews," Sept. 1957, *Proc.* V. 54, p. 254. The following comments have been received from Mr. Dehousse, the author:

"It is inadequate to state, as mentioned by the reviewer, 'It is not apparent how the flexural theory varies with the external load.' In fact in any shell problem, treated according to the elastic theory, the flexural terms do not depend only upon the load applied over the shell, but also upon the boundary conditions. Thus, unless being very restrictive, it is not possible to give practical tables showing directly how the flexural theory varies with the external load."

A numerical method of analysis for doubly curved shell structures

S. P. Banerjee, *The Indian Concrete Journal* (Bombay), V. 32, No. 1, Jan. 1958, pp. 14-20

Derives theory and presents the application of a numerical method of analysis for an illustrative example to a hyperbolic paraboloid shell roof.

Theory of structural analysis and design

James Michalos, The Ronald Press Co., New York, 1958, 552 pp., $12

Theory and method of analysis for indeterminate structures. Particularly suitable as a textbook for graduate courses in structural analysis. It will also be a convenient source of information on analysis of unusual structures as a reference for practicing engineers. Exact and approximate procedures are included. Classical and modern numerical methods are described and illustrated by examples. The particular subjects covered include continuous beams, a very complete treatment of arches with extensive influence lines and tables for both vertical and horizontal loading on single or continuous arches and arches with ties, secondary stresses, buckling, space frames, curved structures loaded normal to their plane, and complex rigid frames.

Simplified design of reinforcement in concrete structures under eccentric loading (in Slavonian)

Srdan Turk, *Gradbeni Vestnik* (Ljubljana), No. 45-46, 1956-57, pp. 99-107

Reviewed by J. J. Polivka

Author's original method of calculating both reinforcements is accompanied by graphs and explained in various examples, based on determination of certain point T corresponding to the specific eccentricity and loading. The effect of buckling is also considered.

Concrete silos

T. J. S. Mallagh, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 84, Bulletin No. 4, Mar. 1958, pp. 123-143

A discussion of analysis, design, specifications, and construction of industrial concrete silos for storage of bulk solids. Reviews various theories for the design pressure, both static and dynamic. Considers the economics of cylindrical or prismatic cross section shapes and various systems of construction, including slip form, precast, and prestressed.

Improved angle-balancing method for highly statically-indeterminate frames (Verbessertes Drehwinkel-Iterationsverfahren für hochgradig statisch unbestimmte Rahmen)

H. Craemer, *Der Bauingenieur* (Berlin), V. 32, No. 3, Mar. 1957, pp. 76-77

Reviewed by Aron L. Mirsky

Basic equation of slope-deflection method, without sidesway,

$$M_{ik} = FEM_{ik} - 2\frac{EI}{L}(2\phi_i + \phi_k)$$

is transformed into an expression better suited to iterational procedures. Numerical work is done on sketch of structure, as in Hardy Cross' moment-distribution method, but is

said to be less subject to computational error than is moment distribution. Special cases, such as members with one end fixed, are easily accommodated. Two worked examples illustrate method.

In June 1957 issue, p. 235, Ernst Melan further discusses and compares both methods.

Analysis of rigid frameworks subject to sidesway (in Rumanian)

I. HARGHEL, *Industria Construcţiilor si a Materialelor de Construcţii* (Bucharest), V. 8, No. 7, July 1957, pp. 388-398

Reviewed by J. J. POLIVKA

Method of elastic deformations by solving equations in which groups of simultaneous rotations at several rigid joints are redundant. Author's method is applied to numerical examples of frames with multiple bays and floors and also to continuous frames with pitched girders.

Elastic end-restraint of straight flights of stairs in girderless landing slabs (Die elastische Einspannung gerader Treppenläufe in trägerlosen Podestplatten)

LOTHAR FUNKE, *Der Bauingenieur* (Berlin), V. 32, No. 3, Mar. 1957, pp. 90-92

Reviewed by ARON L. MIRSKY

Rigorous analysis, with effects of flexural and torsional stiffness of the landing slabs included, is shown to lead to more economic use of reinforced concrete than usual analysis. Two cases are investigated: landings freely supported at both sidewalls only, and landings supported on all three walls.

Free vibrations of cylindrical shells and of rectangular plates of uniform thickness (Vibrations libres des viole cylindriques et des plaques rectangulaires d'épaisseur uniforme)

JEAN LARRAS, *Travaux* (Paris), V. 42. No. 281, Mar. 1958, pp. 180-182

AUTHOR'S SUMMARY

A study of the behavior of a unit mass of the surface of a shell under the action of the components of free vibrations considered as unit loads. By means of a transformation using Fourier series, calculation of strains reduces to the solution of a system of three homogeneous linear equations with three unknowns. Shows how to calculate the period of free vibration.

Design of eccentrically loaded columns by the load factor method. Parts 3 and 4

J. D. BENNETT, *Concrete and Constructional Engineering* (London), V. 53, No. 3, Mar. 1958, pp. 119-128; No. 5, May 1958, pp. 201-211

Part 3 gives design charts of eccentrically loaded columns with unsymmetrical cold worked reinforcement according to the method recommended in the "British Standard Code of Practice No. 114" (1957). Includes the cases of failure in compression with all the steel on the tension face; with all the steel on the compression face; or with various steel distributions and failure in tension with all the steel on the tension face; compression failure with the whole section in compression; or balanced failure with variable steel distribution. Parts 1 and 2 were reviewed in "Current Reviews" section of the ACI JOURNAL, Feb. 1958, p. 712, and Apr. 1958, p. 910.

Part 4 deals with unsymmetrical mild steel reinforcement. Each part presents design curves for failure in compression or failure in tension. Complete with examples illustrating their application.

Elastic instability of arches under stresses and deformations in their plane (in Italian)

G. CERADINI, *Giornale del Genio Civile* (Rome), V. 94, No. 10-11-12, 1956; reprinted by State Polygraphic Institute, Rome, 1957, 132 pp.

Reviewed by J. J. POLIVKA

Modern use of high strength structural materials has made thorough knowledge of elastic instability very important. Thorough analysis is presented of arches with one, two, and three hinges, restrained, circular, parabolic, with constant and variable moment of inertia. First part deals with application of static criterion, influence of axial deformation. Part II covers energy criterion applied to parabolic arches of second and fourth degree with constant and variable moment of inertia, subject to concentrated and uniform loading, symmetrical and assymmetrical deformation.

Part III discusses the title subject under consideration of the elastic theory of the second order. This part includes definitions of fundamental hypotheses, influence of deformations on forces to which arch elements are exposed, virtual work of these forces and their potential energy, relations of orthogonality,

solution methods of equations of the indifferent equilibrium, and solution of the elastic problem of second order.

Materials

TVA uses nonspecification fly ash

G. K. Leonard and P. A. Schwar, *Civil Engineering* V. 28, No. 3, Mar. 1958, pp. 58-62

Presents research data giving the properties of concrete made using TVA fly ash as an admixture. This mechanically collected fly ash is considerably coarser than fly ash meeting ASTM requirements for fineness. Concrete containing this fly ash performed about as expected. Several projects using this concrete are described.

Pozzolana investigations with special reference to acid attack

F. V. Doidge, *New Zealand Engineering* (Wellington), V. 12, No. 12, Dec. 15, 1957, pp. 420-421
Highway Research Abstracts Apr. 1958

Discusses the problem and effects of sulfuric acid attack in sewers, the method of generation of the acid on the sewer walls, and methods for prevention of, or increasing resistance to attack. A suggested method of increasing resistance is by the use of pozzolans, the derivation and effect of which are described. Experiments showed that the use of a naturally occurring pumicite increased the resistance of portland cement concrete to attack by sulfuric acid up to concentrations of 2 percent.

Structure and physical properties of hardened portland cement paste

T. C. Powers, *Journal of the American Ceramic Society*, V. 41, No. 1, Jan. 1958, pp. 1-6
Author's Summary

Methods of studying the submicroscopic structure of portland cement paste are described, and deductions about structure are presented. The main component, cement gel, is deposited in water-filled space within the visible boundaries of a body of paste. Space filled with gel contains gel pores; space not filled by gel or other solid material is capillary space. Hygroscopicity of cement gel, and capillary pores, accounts for various aspects of the properties and behavior of concrete. Data on gel and paste structures are used in discussing strength, permeability, volume stability, and action of frost.

Transactions of the conference on the chemistry of cement (Trudy Soveshchaniya po Khimii Tsementa)

P. P. Budnikov, Yu M. Butt, S. N. Royak, and M. O. Yushkevich (editors), Scientific Technical Society for the Construction Material Industry, Moscow, 1956, 488 pp. 24R25K
Ceramic Abstracts Mar. 1958 (Williams)

This book is a compilation of papers on portland cement. The first paper by Budnikov, gives the usual introductory review of the contributions of Soviet science to the chemistry of cement. Thirty reports which follow cover chemical and physical properties of cement, methods of testing these properties, effects of some admixtures, and certain aspects of cement manufacture.

Studies in cement-aggregate reaction. Chapter 26—Comparison of the effect of soda and potash on expansion

C. E. S. Davis, *Australian Journal of Applied Science* (Melbourne), V. 9, No. 1, Mar. 1957, pp. 52-62

Reports on laboratory investigation of the effect on opaline mortar bars of four sources of alkali: potassium hydroxide in mixing water, potash in cement minerals, sodium hydroxide in mixing water, and soda in cement minerals. Concludes that the expansion of mortar made from a reactive aggregate depends mainly on total alkali content of the cement, much less on which individual alkali predominates, and still less on how rapidly the alkali dissolves. In one group of comparable mortars, however, a smaller amount of soda than of potash caused the same expansion. Preceding chapter was reviewed in "Current Reviews" section, ACI Journal, Jan. 1958, p. 631.

Destructive alkali-aggregate reaction in concrete

Technical News Bulletin, National Bureau of Standards, V. 41, No. 12, Dec. 1957, pp. 199-200
Highway Research Abstracts Apr. 1958

Besides measuring the pressure developed in the reaction, the study also considered the role of the physicochemical properties of the aggregates in concrete disintegration. Readings over a period of several months indicate pressures above 2000 psi, which is greatly in excess of the tensile strength of concrete.

The bureau found that alkalies alter several of the physicochemical properties of aggregates. Alkalies may either attack the sur-

face of the aggregate or cause appreciable swelling. Investigations on the hygroscopicity of the aggregate materials were conducted to determine the role of water in the alkali-aggregate reaction. Water is not only required for this reaction but is one of the chief constituents of the products resulting from the reaction. A possible correlation was found between hygroscopicity and destructive expansion in various materials.

Sydney fly ash in concrete. Part 2

G. B. WELCH and J. R. BURTON, *Commonwealth Engineer* (Melbourne) V. 45, No. 6, Jan. 1958, pp. 62-67

AUTHORS' SUMMARY

Results of an experimental investigation into the suitability of several Sydney fly ashes for use in concrete construction are presented. In the first part (see "Current Reviews" section of the ACI JOURNAL, Jan. 1958, p. 912) details of the tests carried out were given, followed by preliminary discussion of results. In this part, results are examined in detail.

Cauldon cement works

The Engineer (London), V. 204, No. 5307, Oct. 11, 1957, pp. 529-532

Cement with less fuel. The semidry process at Cauldon

Engineering (London), V. 184, No. 4781, Oct. 25, 1957, pp. 535-537

Reviewed by ARON L. MIRSKY

Description of new cement works at Cauldron Low. Plant, first of this type in England, currently with one kiln but with provision for future expansion, has an annual capacity of 200,000 tons. Up to 40 percent saving in kiln fuel is claimed for semidry process as compared to wet process.

Trend of development of aggregate preparation plants for producing concrete on construction sites (Die Entwicklungsrichtung der Aufbereitungsanlagen für Zuschlagstoffe für die Betonherstellung auf Baustellen)

BRUNO ZORBACH, *Der Bauingenieur* (Berlin), V. 32, No. 4, Apr. 1957, pp. 135-142

Reviewed by ARON L. MIRSKY

Good survey of the criteria for economical plant for quantity production of aggregate for projects, such as hydroelectric plants, dams, locks, etc., requiring high yardage of concrete, and of the modern equipment now available to satisfy these criteria. It is interesting to note that some portable crushing equipment of American origin is included.

Paint research and technology—V. 2

Engineering Progress at the University of Florida, V. 11, No. 12, Dec. 1957; *Bulletin* No. 94, Florida Engineering and Industrial Experiment Station, 156 pp., $4

Includes 20 papers on paint technology. Three of the papers are particularly applicable to painting concrete masonry and concrete. They present results of research and actual performance of styrene-butadiene latex paints, polyvinyl acetate emulsion paints, and acrylic resin emulsion paints for concrete. The book should be useful as a supplement to the ACI committee report "Guide for Painting Concrete," particularly since it provides more complete and more up-to-date information on the three types of paint noted.

Cement and clay grouting of foundations: The use of admixtures in cement grouts

ALEXANDER KLEIN and MILOS POLIVKA, *Proceedings*, ASCE, V. 84, No. SM1, Part 1, Feb. 1958, pp. 1547-1 to 1547-24

AUTHORS' SUMMARY

The theoretical and practical aspects of the use of admixtures are discussed with respect to grout mixtures of various compositions. Types of admixtures which are useful in grouts, with a brief discussion of the mechanisms involved, are described, as are also test equipment and methods of test for evaluating the contribution of admixtures to grout quality. Typical relationships among grout properties are illustrated.

1957 supplement to book of ASTM standards (including tentatives). Part 1—Ferrous metals

American Society for Testing Materials, Philadelphia, 509 pp.

This 1957 supplement to Part 1 on ferrous metals contains the revised standards and the new and revised tentatives in these materials fields that have been accepted since the appearance of the 1956 supplement to book of ASTM standards. Seventeen of the 21 standards are replacements of existing standards; similarly, of the 77 tentatives, 52 are replacements of existing tentatives; 10 represent standards revised and reverted to tentative, while 15 are published for the first time. Four standards and 11 tentatives

with minor revisions, and 4 tentatives adopted as standard in 1957 without revision are listed in the contents but are not published in this supplement. Also listed in the contents are two emergency alternate provisions which have been withdrawn.

1957 supplement to book of ASTM standards (including tentatives). Part 3—Cement, concrete, ceramics, thermal insulation, road materials, waterproofing, soils

American Society for Testing Materials, Philadelphia, 412 pp.

This supplement contains the extensively revised standards and the new and extensively revised tentatives in these materials fields that have been accepted since the appearance of the 1956 supplement to the 1955 book of ASTM standards. Fifteen of the 17 standards published in this supplement are replacements of existing standards; one standard represents a tentative revised and adopted in 1957; one standard represents a tentative revised and adopted in 1957; and one standard represents a tentative adopted as standard without change in substance, but with extensive editorial revisions.

Precast Concrete

Precast reinforced concrete roof trusses for public housing schemes (in Hebrew)

A. Alweyl, J. Fichmann, and J. Karni, *Research Paper* No. 5, Building Research Station, Technion, Israel Institute of Technology, Haifa, 1957, 90 pp.

Results of a survey in research on the manufacture and use of precast trusses for pitched roofs of residences in Israel. The various types of trusses manufactured and available are described as well as tests performed on these units. Methods of the prescribed testing procedures are given and recommendations for production outlined.

Concrete products manual

Florida Concrete and Products Association, Winter Park, Fla., 1957, 224 pp., $15

A brief but carefully edited loose-leaf technical data and marketing reference handbook to the concrete products available in Florida. The book features four sections: concrete masonry, ready-mixed concrete, prestressed concrete products, and precast concrete products. Technical suggestions for specifications, technical data, cost data, and availability of products in each field are furnished. All fields are well described by a large number of excellent illustrations.

Installation and mechanical equipment for precasting large structural slabs (in Rumanian)

M. Luca, *Industria Constructiilor si a Materialelor de Construcții* (Bucharest), V. 8, No. 7, July 1957, pp. 384-387

Reviewed by J. J. Polivka

Description of the prefabrication plant for slabs cast of concrete with volcanic ash and ceramic aggregates, installed on the construction site of a huge student dormitory.

Precast large concrete slab houses in Czechoslovakia (Velkopenelové obytné domy v ČSR)

Svetla Francu, *Stavebnícky Časopis Slovenskej Akadémie Vied* (Bratislava), V. 6, No. 6, Dec. 1957, pp. 337-353

Reviewed by J. Stork

Evaluation of results and experiences with three different types and constructional systems of multistory concrete houses assembled on site from large precast concrete slabs. Considered only types which have passed the experimental and semiproduction stage.

Prestressed Concrete

Fire resistance of prestressed concrete

A. W. Hill, *Constructional Review* (Sydney), V. 31, No. 2, Feb. 1958, pp. 24-31

Summarizes the results of research on fire resistance carried out in Great Britain and draws conclusions from which tentative recommendations may be made for code requirements. Gives brief details of the standard test and a discussion of the general behavior of various types of prestressed structural elements.

New system of prestressed silos

W. J. Petri, *Cement* (Amsterdam), V. 9, No. 9-10, Oct. 1957, pp. 386-388

Reviewed by John W. T. Van Erp

Walls are built of radial shaped cement blocks. A prestressing reinforcement of single steel wires (about 8 in. on centers) is wrapped around the walls and is prestressed in a simple way by pulling two wires together with mild steel tie wires at many points on the perimeter. The tension in these tie wires is a

measure of the tension in the prestressed steel, and can be easily measured. Protective cover 1¼ in. thick is sprayed on. The system does not involve any expensive apparatus and is well within the capabilities of the average small builder.

Prevention of crack formation in cylindrical tanks (in Dutch)

G. W. P. Van Der Heiden, *Cement* (Amsterdam), V. 8, No. 21-22, Oct. 1956, p. 526; V. 9, No. 3-4, Apr. 1957, pp. 139-142

Reviewed by John W. T. Van Erp

Article concludes that prestressed reinforcement is only definite means of crack control for bigger tanks. Influence of cantilevered ring platforms is analyzed, also use of prefabricated anchor blocks. Examples of sewage treatment tanks are given as well as repair of cracked tanks by means of post-tensioned cables.

Shell construction of the "Hydraulic Engineering Hall" of the Darmstadt School of Technology (Die Schalenkonstruktion der "Wasserbauhalle" der T. H. Darmstadt)

A. Mehmel and H. J. Wittneben, *Der Bauingenieur* (Berlin), V. 32, No. 2, Feb. 1957, pp. 46-48

Reviewed by Aron L. Mirsky

To provide 25 x 70 m of columnless space to accommodate the flumes, channels, and other experimental facilities of the new *Versuchsanstalt für Wasserbau*, roof was constructed as series of seven flat prestressed barrel vaults 10 x 25 m in plan, supported only at the ends of their intersections by cranked columns. Shells were prestressed to reduce cracking and consequent danger of corrosion of reinforcing due to damp atmospheric inside structure. For architectural reasons, end walls are constructed above the shells.

Prestressed insulated concrete roofs for heated industrial plants (in Rumanian)

I. P. Mihai, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 7, July 1957, pp. 375-384

Reviewed by J. J. Polivka

Study of special types of composite roof structures consisting of typical prestressed joists combined with lightweight insulating slabs interconnected with stirrups. Design is thoroughly discussed and analyzed for standard top slabs 30 x 60 in. with 4 inside holes at distances of 15 and 30 in., 30 in. being the spacing of the supporting joists. Nine different types are investigated for spans from 7 to 20 ft with tabulated results, amply illustrated, with all characteristic properties, especially thermal insulation, bearing capacity and waterproofing. Good results were obtained with slabs of cellular concrete cured under steam pressure. Various details of periphery joints and their grouting methods are described. Calculation is demonstrated by examples.

Properties of Concrete

Elastic and permanent compression of concrete in short-time tests and the possibilities of influencing it (in German)

J. Bonzel, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 10, 1957, pp. 421-30

Ceramic Abstracts Apr. 1958

Discusses the precautions to be taken in determining the relation between the modulus of elasticity in compression and compressive strength and the factors that affect it, such as storage conditions, temperatures, kind of sample, porosity, and water-cement ratio. Diagrams show the change in the modulus of elasticity and the compressive strength as a function of these factors in rapid test.

Field soniscope tests of concrete, 1953-1957 tests

E. C. Roshore, *Technical Memorandum* No. 6-383, Report 2, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Mar. 1958, 14 pp., $0.50

Author's Summary

In 1957, pulse velocities were determined by soniscope testing at 160 stations in four dams (Clark Hill, Wolf Creek, Bull Shoals, and Norfolk), one lock (Tuscaloosa), and specimens at a field exposure station established in Georgia by the Portland Cement Association. Data obtained are to be used as a basis for evaluating subsequent changes in the concrete at the test locations, and will aid in the long range evaluation of the significance of data obtained from the soniscopic examination of concrete structures.

The data were compared with similar data obtained in 1953 and 1955. The results suggest that changes in the concrete measurable by pulse velocity, at all but four stations, during the period 1953-1957 have been of

minor magnitude and do not permit conclusions as to the general trend of the concrete to increase or decrease significantly in quality.

The concrete at four stations in Tuscaloosa Lock, Monolith 20, increased significantly in velocity. This is assumed to result from a filling up of the cracks in this monolith with an alkali-aggregate reaction product.

Influence of cement on creep of concrete and mortar

A. M. NEVILLE, *Journal of the Prestressed Concrete Institute*, V. 2, No. 4, Mar. 1958, pp. 12-18

Considers the effect of type (chemical composition), burning, and fineness of cement on creep of concrete and mortar in compression and in tension. Essentially a progress report of laboratory research which is continuing. The principal conclusion is that creep appears to be inversely proportional to the rapidity of hardening of the cement used. Includes 17 references on creep.

Structural Research

Strain measurement by means of electromagnetic torsional strain gages (Meranei deformácií magnetoelektrickými torznými tenzometrami)

TIBOR JAVOR, *Stavebnícky Časopis Slovenskej Akadémie Vied* (Bratislava), V. 6, No. 6, Dec. 1957, pp. 375-382

Reviewed by J. STORK

Description and results of strain measurements of concrete beams and skew slabs made with new type of electromagnetic straingages designed on the basis of the Straka-Wiedemann effect and developed in Czechoslovakia in 1953-1956.

Experimental analysis of steel penstocks embedded in concrete in a rock tunnel (in Slavonian)

BORIS VEDLIN, *Gradbeni Vestnik* (Ljubljana), No. 45-46, 1956-57, pp. 67-73

Reviewed by J. J. POLIVKA

Measurements were made in the hydroelectric plant Jablanica and Moste built in 1951-56, where tunnels of 15 and 17 ft inside diameter were used. Measurements of deformations and stresses were made with electric underwater strain gages. Results are presented in tables and diagrams. It was found that under interior pressure the steel pipe encased in concrete and placed in a fairly solid rock carries only a small part of the pressure, the greater part being resisted by embedding concrete, 4 to 5 ft thick, which fills the space between steel pipe and solid rock.

Hydraulic machine for testing model concrete shells

A. S. HALL, *Constructional Review* (Sydney), V. 30, No. 12, Dec. 1957, pp. 22-25

Describes a method for testing shells under uniform (radial) loads. The shell is positioned upside down against a rubber membrane covering a steel tank which can be filled with water. The shell is bolted down by rigid supports across diaphrams. Load is varied by varying the pressure of the water beneath. Strain readings are taken by electrical resistance strain gages mounted on the specimen.

General

Building construction handbook

FREDERICK S. MERRITT (editor), McGraw-Hill Book Co., New York, 2nd Edition, 1958, 906 pp., $15.00

In 29 sections, the book offers basic information in all phases of building design and construction. It is designed as a practical aid for engineers, construction people, and management for making decisions affecting design, materials, and construction methods. Each section is prepared by a specialist in the field. Subjects treated include legal aspects, properties of various building materials, stresses, soils, concrete construction, structural and lightweight steel construction, and wood construction, heating and air-conditioning problems, water supply and waste disposal, electrical installations, surveying, cost estimates, specifications, and insurance.

Evaluation of electric meters for determining moisture content of fine aggregate

J. M. POLATTY, *Technical Report* No. 6-476, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Mar. 1958, 22 pp., $0.50

AUTHOR'S SUMMARY

Three commercially available moisture meters were tested in both a laboratory and a field evaluation program to determine their ability to indicate accurately and instantly the percentage of surface moisture in sand. Tests were made using three gradations each

of a natural and a manufactured sand. In addition, one of the meters was tested at a field installation in the regular production of concrete for a period of approximately 2 years.

The test results indicate that the meters will show a change in the moisture content of the sands tested, that periodic calibration of the meters is necessary to maintain their accuracy, and that when properly used, they will assist in maintaining production of concrete of uniform moisture content.

Fundamental and practical concepts of soil freezing

Bulletin 168, Highway Research Board, 1957, 205 pp., $4.00

The nine reports in *Bulletin* 168 add to the published information of a practical nature on theories and hypotheses concerning the fundamentals of the soil-water freezing phenomenon. The papers deal with the practical aspects of the relative thermal conductivities of different soils and pavements as they influence soil temperatures; the ability of engineers to compute pavement temperatures from data contained in weather reports; and loss and recovery of bearing capacity of a wide range of soil types.

Measures have been made of the relative amounts of frozen and unfrozen water in various types of soils, and the influence of that water on the properties of the soils. A discussion of available knowledge concerning thermal conductivity of soils and similar granular systems is included.

Concrete—Judgment and inspection using the Schmidt sclerometer (Concreto—Julgamento e inspecção pelo esclerometro Schmidt)

A. A. Caldas Branco, Edition A. A. Caldas Branco, Série Divulgação, Rio de Janeiro, 1956, 100 pp.
Reviewed by J. Laginha Serafim

The author explains nondestructive tests using two models of the Schmidt sclerometer (rebound test). Rules necessary to obtain representative data from rebound tests on 6 x 12-in. concrete cylinders are suggested. Equations relating experimental data on the compressive strengths and the indications of the apparatus are given for the upward, horizontal, and downward tests. Tables with results of tests performed are given.

Ten years of building research 1947-1957

National Research Council of Canada, Division of Building Research, Ottawa, NRC 4435, Sept. 1957, 120 pp., $1

Discusses and illustrates some of the tests and testing methods carried out by the Division of Building Research on materials and constructions, and for problems encountered in Canadian building. Building materials, building design, soils, snow and ice problems, and fire research are some of the particular problems which have been studied. Includes a list of the publications available from the Division of Building Research as well as technical papers which have been presented but are not yet available as publications, including bibliographies and translations.

Demolition by controlled blasting

The Engineer (London), V. 204, No. 5306, Oct. 4, 1957, p. 489
Reviewed by Aron L. Mirsky

It is perhaps an intrusion to review an item such as this in a journal devoted to construction, yet the occasion for removal of existing structures arises—sometimes even resulting in an improvement—and the subject matter is thus of interest as well as importance.

Methods used for demolition of heavily reinforced concrete structures, in the center of London, including a "skin" wall for a wartime shelter, are described.

Tentative method for the determination of the original water-cement ratio of hardened concrete

A. W. Brown, *Journal of Applied Chemistry* (London), V. 7, Pt. 10, Oct. 1957, pp. 565-572
Highway Research Abstracts Apr. 1958

Method involves the determination of the capillary porosity of a dried sample of concrete by refilling it with an organic liquid, measuring it as an increase of weight, and calculating the equivalent amount of water. Making an allowance for any carbonation which has taken place, the original water-cement ratio can be calculated. Experiments show that errors which might arise due to air entrainment, accidental or deliberate, are negligible. Possible effects of porous and calcareous aggregates are discussed.

ACI News Letter

Vol. 30, No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE July 1958

Panama City apartment house . . . p. 3

155 Reasons Why...

What is the purpose of air entrainment in concrete?

How can durable aggregates be assured?

How do the requirements of placing affect the proportions of aggregates and cements?

. . . . The ACI CONCRETE PRIMER should be your guide to better concrete. The PRIMER develops in simple terms the principles governing concrete mixtures and presents a handy reference text for those who apply these principles to the production of permanent structures in concrete. Expanded to 72 pages—155 answers—the new pocket-size edition of the CONCRETE PRIMER brings developments of the past three decades into the question-and-answer format of this long-popular handbook.

$0.50 TO ACI MEMBERS NONMEMBERS $1.00

American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Michigan

Please send_____copies of the 1958 edition of the ACI Concrete Primer.
($0.50 for ACI members, $1.00 for nonmembers)

A check (or money order) for_____is enclosed.

Name.....

Address.....

City.....Zone.....State.....

ACI

News Letter Contents

July 1958

On the Cover—Reinforced concrete apartment building recently erected in Panama City illustrates wide use of concrete in construction in Central America. Located in a residential section called "Campo Alegre," this is one of the tallest apartment buildings in the Republic of Panama. Calvin L. Stempel and Diego E. Pardo, Jr., both of Panama City, were architect and structural engineer respectively for this project. The contracting firm was Stempel and Garcia.

Bylaws Revised..... 3

ACI Bylaws..... 4

Positions and Projects..... 9

Contemporary Concrete Church.....10

Looking Ahead.....13

Cement Plant Expansion.....17

Who's Who.....19

Committee 208.....19
Committee 325.....19
Committee 617.....19
Committee 711.....20
Richard C. Mielenz, Vladimir E. Wolkodoff, James E. Backstrom, and Harry L. Flack....20
Donald D. Meisel, Cyril D. Jensen, and Walter H. Wheeler.....23
I. D. MacKenzie.....23
Fazlur R. Khan and Andrew J. Brown.....24

Honor Roll.....27

New Members.....27

Erratum.....28

Tools, Materials, Services.....31

List of Advertisers.....34

bylaws revised

local chapter organization provisions now approved

ADDITIONS AND CHANGES in the Bylaws of the American Concrete Institute which were adopted at the 54th annual convention in Chicago last February have been ratified by letter ballot of Institute members, and became effective June 1. Major modification is the addition of a section on local chapters, which authorizes, defines, and limits ACI's organization at the local level. One such group, the Southern California Chapter, formed provisionally last year, now assumes official status. Regional groups in Oklahoma (see p. 9 of this News Letter) and elsewhere are now tentatively planning local chapters.

Changes in terminology regarding members were approved in Article I of the Bylaws, and some modification has been made there regarding transition from Student grade to other membership grades.

Changes regarding officers, elections

In the section on officers, an added provision establishes a vacancy on the Board of Direction whenever a director is elected president or vice-president before expiration of his director's term. Some modification was also made in the method of canvassing ballots for officers at the annual convention, and in the provisions on elegibility of officers for re-election.

The revised Section IV omits mention of the Secretary-Treasurer. This is to conform to intent of 1952 revision which removed him from the Board of Direction and made the term of office indefinite.

In Article III the method of notification of meetings has been more definitely stated. The two final articles have been renumbered to allow for the insertion of Article V on local chapters.

Complete text of the revised Bylaws appears on the following pages.

Bylaws*

ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

Sec. 2. An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinary meritorious service to the Institute. He shall have the same rights and privileges as a Member, but shall not be subject to dues.

The total number of Honorary Members of the Institute and the maximum number to be elected in any one year shall be at the discretion of the Board of Direction.

The Committee on Honorary Membership shall consist of the three Past President members of the Board of Direction. The chairmanship of this committee shall be vested each year in that person beginning his third-year term as a Past President member of the Board. This group shall serve as a screening committee for the formulation of recommendations to the Board. If one or more members of the regularly constituted committee on Honorary Membership is deceased or inactive, the President shall appoint to each vacancy some regular member of the Board of Direction to serve through the period of shortage, the Senior Past President non-appointed member of the committee functioning as its chairman.

The Committee on Honorary Membership is expected to be continuously alert for deserving prospects and open to considered suggestions from all sources. It shall submit a formal report to the President of the Institute each year at least thirty days prior to the Fall meeting of the Board. The annual report is mandatory even though there be no recommendations.

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency of government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

Sec. 4. All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversary of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 6. Applications for and resignations from membership and requests for change of representatives of Corporation or Contributing Members shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

*Latest amendment adopted at the 54th annual convention Feb., 1958, and ratified by letter ballot of the membership June 1, 1958.

ARTICLE II—OFFICERS

Section 1. The officers shall be a President, two Vice-Presidents, twelve Directors, the three latest living Past Presidents who continue to be Members of the Institute, and the Secretary-Treasurer. The President, the Vice-Presidents and the Directors shall be elected from the Institute membership. When a Director is elected Vice-President or President, his office as Director shall become vacant. The Secretary-Treasurer shall be appointed by the Board of Direction.

Sec. 2. The President, Vice-Presidents, Directors and the three latest living Past Presidents who continue to be Members, shall constitute the Board of Direction.

Sec. 3. Before September 1 of each year the Committee on Nominations shall report to the Secretary-Treasurer of the Institute the candidates nominated for offices to become vacant at the next annual convention and 20 candidates for membership on the Committee on Nominations which is to serve in the following year. In the selection of candidates for Directors, the Committee on Nominations shall have due regard for diversity of professional and geographical representation. Each candidate for Board membership must have given consent to his nomination before the report is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days prior to the next ensuing annual convention. By petition to the Board of Direction signed by at least ten members of the Institute, within 30 days thereafter, additional nominations for offices or for membership on the Committee on Nominations may be made.

The complete list of nominations shall be submitted 60 days before the next annual convention to the Institute membership for letter ballot to be canvassed on the first day of the convention and the result announced at a session of the convention on the second day. The candidate for any office receiving the most votes shall be declared elected and the candidate receiving the most votes for membership on the Committee on Nominations shall be chairman of that committee; the four next highest shall be declared elected members of the committee. With these five the three past-president members of the Board of Direction shall serve, making a total membership of eight.

Should any member of the Committee on Nominations thus chosen fail, within 15 days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the five elected places on the committee shall be filled.

Sec. 4. Terms of office shall be as follows: President, one year; Vice-President, two years, with one Vice-President elected each year; Directors, three years, with four Directors elected each year. A year is here construed as the period between the reports of tellers on canvass of ballots for Board members at two successive annual conventions.

Sec. 5. A President, Vice-President, or Director, having served a full term after being elected to that office, shall be ineligible for re-election to the same office until the lapse of at least one year.

Sec. 6. The term of each officer shall begin immediately upon the announcement by the tellers of the canvass of ballots by which he is elected and shall continue until a successor is elected.

Sec. 7. A vacancy in the office of President shall be filled by the Vice-President having seniority in that office.

Sec. 8. Vacancy in any office shall be filled by appointment by the Board of Direction for the unexpired term, except as provided in Section 7.

Sec. 9. In the event of disability of any officer of the Institute, or neglect in the performance of the duties of the office, the Board of Direction shall declare the office vacant.

Sec. 10. The Board of Direction shall have general supervision of the affairs of the Institute. It shall authorize and appoint the chairmen of such administrative and technical committees and assign to them such duties and such authority as it deems needful to carry on the work of the Institute. Additional committee members shall be appointed by the President.

Sec. 11. There shall be an Executive Committee of the Board of Direction consisting of the President and three of its members appointed by the Board of Direction.

Sec. 12. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 13. The President shall perform the usual duties of the office. He shall preside at the annual convention, at the meetings of the Board of Direction and of the Executive Committee, and shall be ex officio member of all committees. He may name a chairman to serve in his place for any sessions of the convention.

The Vice-Presidents, each in the order of his seniority in that office, shall discharge the duties of the President, in his absence. In the absence of President and both Vice-Presidents, a President Pro Tem, appointed by the Board, shall discharge such duties.

Sec. 14. The Secretary-Treasurer shall perform such duties, furnish such bond and receive such salary as shall be determined by the Board of Direction.

ARTICLE III—MEETINGS

Section 1. The Institute shall hold an annual convention and such other meetings as may be authorized by the Board of Direction. The time and place of all meetings shall be fixed by the Board of Direction. Notice of this action shall be sent to all members at least 30 days previous to the date of each meeting. Publication in the Institute JOURNAL shall be construed to be adequate notice.

Sec. 2. The Board of Direction shall meet at least twice each year at the time and place fixed by the Board.

Sec. 3. The Executive Committee shall meet on call of the President or of any three of its members.

Sec. 4. Twenty-five members shall constitute a quorum for meetings of the Institute; eight members shall constitute a quorum for meetings of the Board of Direction; and three members for meetings of the Executive Committee.

ARTICLE IV—DUES

Section 1. Dues shall be payable in advance on the first day of the month of notification of the Member applicant of his election by the Board of Direction, and annually thereafter, as follows: Contributing Members, $100.00; Corporation Members, $50.00; Members (individuals) in North America* and U. S. possessions, $15.00; Members elsewhere, $12.00; Junior Members, $7.50; Student Members, $5.00. Corporation Members outside North America may upon application be granted a 20 percent reduction in dues. Any individual Member may be admitted to Life Membership upon payment of a sum determined by the Executive Committee based on 90 percent of the membership dues as established at the time of application, credited with 3 percent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

Sec. 2. A Member of any grade shall be entitled to receive one copy of each issue of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE published in the period of his membership and additional or other publications as determined by the Board of Direction.

*Includes all countries and territories in Central America and West Indies. Authorities: *Commercial Atlas of America*, Rand McNally & Co., Chicago; and *Survey Atlas of the World*, The Times, London.

Sec. 3. A Member in any classification whose dues remain unpaid for a period of two months shall forfeit the privileges of membership and shall be sent notice to this effect by the Secretary-Treasurer. A Member in any classification whose dues are unpaid for a period of one year from their due date shall have his name stricken from the rolls unless otherwise specifically ordered by the Board of Direction. Members may be reinstated upon payment of all indebtedness against them on the books of the Institute and dues for one year in advance of that date.

ARTICLE V—LOCAL CHAPTERS

Section 1. Local chapters to provide a means of advancing the interests of the Institute in a specified geographical area and of furthering the chartered objectives for which the Institute is organized may be authorized, from time to time, by the Board of Direction upon such terms and conditions as it may deem advisable.

Sec. 2. The Board of Direction shall define the boundaries of chapter areas, within or without the confines of the United States, in which the local chapter shall be authorized to carry on its activities. Boundaries of chapter areas may be changed, from time to time, by the Board of Direction at its discretion, provided that prior notice is given to the chapter or chapters involved.

Sec. 3. Organization of a chapter may be authorized by the Board of Direction upon the written request of 25 members of the Institute residing within the proposed chapter area, provided that at least 50 members reside in the area it is proposed to encompass.

The Board of Direction may at any time terminate the existence of any chapter when in its judgment the interests of the Institute make such action desirable.

Sec. 4. Each Institute member shall be deemed eligible to belong to a chapter if his permanent address of record at Institute headquarters is within the confines of that chapter area. A member who does not maintain active membership in the local chapter, as defined by the chapter bylaws shall have no part in the government thereof. Members not resident in the geographical area of a given local chapter may become members thereof if the bylaws of the chapter permit, but shall not be entitled to vote or to hold office in such chapter.

Chapter dues shall not exceed one-half of the Institute dues for corresponding classifications of membership. Chapters may accept contributions to assist in financing activities which are in accord with their approved bylaws.

Sec. 5. Proposed bylaws of local chapters and amendments thereto shall be in conformity with the chartered aims and objectives and also the Bylaws of the Institute and shall be approved by the Board of Direction before becoming effective.

Local chapters shall conduct only such activities as shall conform to and be in accordance with the chartered aims and objectives of the Institute.

Local chapters shall not speak, or attempt to speak for the Institute as a whole in any matter unless specifically so authorized by the Board of Direction; and further shall not incur or attempt to incur financial obligations of any kind binding upon the Institute.

Sec. 6. Each chapter shall annually submit to the Board of Direction a report of its activities and finances. Annual reports by chapters shall reach the Secretary-Treasurer of the Institute not later than January 30 of the following year.

Sec. 7. The President, or an officer of the Institute designated by him, shall make an annual visit to each chapter to confer with its officers on problems related to operation of the chapter.

Sec. 8. Funds of the Institute may be allotted to each chapter on a basis determined by the Board of Direction, the formula adopted being applicable to all chapters. Payment of funds shall be subject to regulations established from time to time by the Board of Direction.

ARTICLE VI—STANDARDS

Section 1. The Board of Direction shall adopt such rules and regulations governing the procedure for the consideration and adoption of Standard Specifications, Standard Practice Recommendations and Standard Definitions as will safeguard their soundness and representative character.

ARTICLE VII—AMENDMENTS

Section 1. Proposed amendments to these Bylaws, signed by at least 15 Members, if presented in writing to the Board of Direction 90 days before the annual convention, shall be mailed to the membership at least 30 days prior to the annual convention. These amendments may be discussed and amended at the annual convention and be passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot canvassed within 90 days after mailing ballot forms shall be necessary for their adoption.

Research conference on plain concrete set for September 10-12

The University of Illinois in cooperation with the American Concrete Institute, National Science Foundation, American Society of Civil Engineers, and the Reinforced Concrete Research Council will hold a conference on fundamental research in plain concrete at Allerton Park, near Monticello, Ill., Sept. 10-12, 1958.

Conference objectives

The objectives of the conference are to remove or push back old frontiers of research and to develop new thoughts and ideas. The major portion of the conference will be devoted to a discussion of future research. The subjects and areas of study which may be considered frontiers of knowledge with respect to concrete research will receive major emphasis. Where it appears feasible, attempts will be made to determine how to achieve a major break-through. The knowledge necessary to understanding the behavior of plain concrete depends upon research in many fields, and so, the conference will be broad in scope. Chemists, physicists, crystallographers, mathematicians and experts in the use of the electron microscope will be present in addition to representatives of organizations from areas of research normally associated with plain concrete.

Some 60 persons have been invited to attend this conference, and the remaining facilities will be available for those who are interested and wish to attend. Because of space limitations anyone interested in attending the conference should write to Professor Clyde E. Kesler, 103 Talbot Laboratory, University of Illinois, Urbana, Ill.

Alpha names four vice-presidents

Norman O. Wagner has been elected senior vice-president of Alpha Portland Cement Co., Easton, Pa., and three other executives have been promoted to vice-presidents. The three new vice-presidents are: Ernest F. Brownstead, Richard L. Rhodes, both of Easton, and Joseph D. Bell of New York.

Intrusion-Prepakt announces appointments

The appointment of John C. King as manager of sales, with headquarters in the Cleveland executive offices, has been announced by Intrusion-Prepakt, Inc. Before becoming chief engineer for Intrusion-Prepakt in 1947, Mr. King was associated with International Engineering Co., Denver, the U. S. Army Corps of Engineers, and the Bureau of Reclamation. He was a faculty member at Robert College in Istanbul, Turkey, for 3 years.

Bruce A. Lamberton has been appointed chief engineer. He has been with the company since 1948 and has completed field assignments ranging from northwestern United States to the gold mines of South Africa. In 1955 he was made manager of research and development for Intrusion-Prepakt.

Positions and Projects

Oklahoma ACI members consider local chapter

Oklahoma City ACI members and concrete industry representatives met May 1 to discuss the possibility of forming a local ACI chapter. Robert P. Witt, Oklahoma A & M College, Stillwater, presided at the meeting. William A. Maples, ACI secretary-treasurer, outlined the history of the development of the local chapter concept, citing the objectives and advantages possible for a local group.

Following discussion, the group took preliminary steps to petition Institute headquarters for the formation of an Oklahoma chapter. A chapter bylaws committee was established with Harold Wenzel, Dolese Co., Oklahoma City, as chairman.

Formation of local chapters within the Institute became a legal possibility for the first time on June 1, when ACI members ratified bylaws changes setting forth the procedure for formation of local chapters. One group, the Southern California Chapter, has already been organized.

Complete details of the bylaws changes are reported on page 3 of this issue of the News Letter.

Viest addresses consulting engineers

Ivan M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill., recently addressed the New York Association of Consulting Engineers at a seminar on composite steel and concrete construction design as applied to buildings and bridges. He discussed the design of composite structures as an example of the practical utilization of research studies.

Mr. Viest has been engaged in experimental and theoretical investigations of composite construction since 1948 and during 9 years of association with the University of Illinois played an active part in development of design methods and specifications for composite structures.

An active member of ACI since 1948, Mr. Viest is currently chairman of ACI-ASCE Committee 333, Design and Construction of Composite Structures.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 116, Nomenclature

Paul W. Gleason
Formigli Corp.
Philadelphia, Pa.

Committee 201, Durability of Concrete in Service

Howard M. Bixby
National Crushed Stone Association
Washington, D. C.

Committee 332, Recommended Practice for Residential Concrete Work

H. T. Gilkey
National Warm Air Heating and Air Conditioning Association
Cleveland, Ohio

Perry H. Petersen
The Master Builders Co.
Cleveland, Ohio

Committee 609, Compaction of Concrete by Mechanical Means

Elmo C. Higginson
Bureau of Reclamation
Denver, Colo.

Committee 616, Coatings for Concrete

Kenneth Tator
Industrial Lining Engineers, Inc.
Edgeworth, Pa.

Committee 714, Concrete Bins and Silos

J. F. Camellerie
Plummer Associates, Inc.
New York, N. Y.

Inside—

Concrete seams joining the panels are heavily reinforced, add to the rigidity of the "space construction," as it is termed by engineer and architect. The rough concrete walls have not been rubbed to a finish, and the wide seams are exposed as a deliberate part of the design. Colored glass set in the openings in concrete panels was made in Chartres, France, by Gabriel Loire, using templates of Mr. Harrison's designs

PRECAST CONCRETE SE

Outside—

Panels of precast concrete, 152 of them, were joined to form a structure half a block long, six stories high. Some panels are triangular, some quadrangular, as much as 35 ft high, weigh up to 11 tons

Two years of unique construction effort were culminated last spring in dedication of the First Presbyterian Church's new home in Stamford, Conn. The moving effect of soaring space and jeweled light associated with the medieval cathedral has been rendered for modern man in contemporary concrete. Reinforced panels were precast, lifted into place, and joined with cast-in-place concrete seams to form a structure 234 ft long, 60 ft high, covering more than a quarter of an acre.

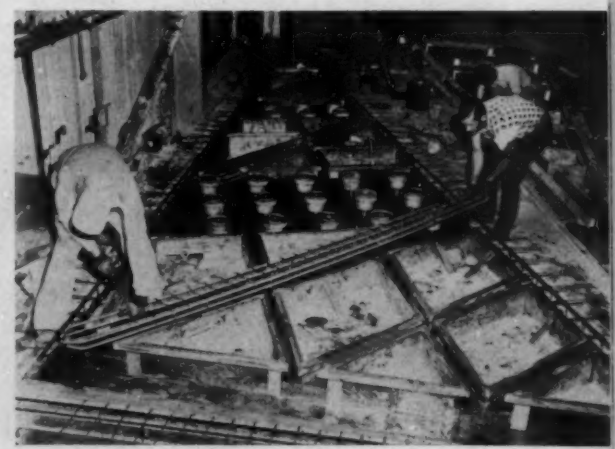
Entirely free of interior supports, structural strength comes in part from the folded slab action of the joined panels, in part from the heavily reinforced seams between the panels which serve as giant ribs reaching from the foundation to lock over the arching roof. Walls of the church sanctuary tilt inward, 74 to 78 deg, and the roof slants at varied angles. Two views of the Stamford church during erection appeared in the July, 1957 News Letter (pp. 1 and 7).

William Harrison, Harrison and Abramovitz, New York, served as church architect, and Felix J. Samuely, London, was structural engineer. Sherwood, Mills, and Smith, Stamford, designed the long, low parish unit which complements the sanctuary. Builders were the Deluca Co., Stamford, and concrete panels were made by Precast Building Sections, Inc.

TE SLABS

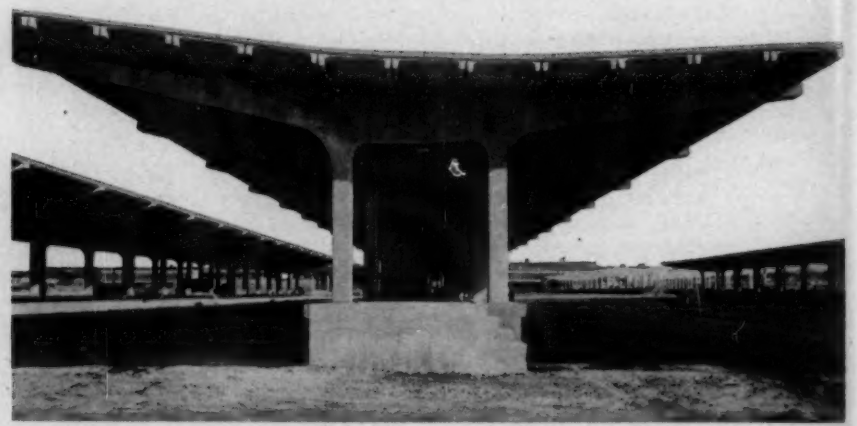
make history in church architecture

Support—
Placing reinforcing bars in one of the openwork concrete panels which was designed to support the inch-thick patterns in colored glass. Bars which bristle around the rim were used to join adjacent panels after they were lifted into place. Seams were filled with cast-in-place concrete



Over 1,500,000 square feet of

PRECAST ROOF CHANNELS "GO TO MARKET"



Only a small part of the huge Farmers Market at Forest Park, Ga., is visible in this photo of a typical farmers' shed. Altogether, the 140-acre Market comprises 32 sheds, 9 dealer buildings, an administration building, and numerous miscellaneous buildings.

Pouring precast reinforced concrete roof channels at job site. A total of 14,783 such channels—ranging in size from 4' x 20' to 4' x 46' with 8" to 16" flanges—and 15,325 tons of precast structural framing, were used in the Market.

● Almost 15,000 precast concrete roof channels went into the buildings for this new 140-acre Farmers Market near Atlanta. In addition, over 15,000 tons of precast concrete members were used in structural framing.

Precast concrete was chosen for its speed and economy in construction, its low maintenance costs, and because it gave the owners a permanent, fireproof market.

In the manufacture of the precast members, Lehigh Early Strength Cement was used to save time and money. Form costs were cut 50%, labor costs 25%, and production time by 50%.

This is another example of the advantages of Lehigh Early Strength Cement in modern concrete construction.

Owner: Georgia Farmers Market Authority, Atlanta, Georgia
Architect: Abreu & Robeson, Inc., Atlanta, Georgia
Engineers: T. Z. Chastain, Atlanta, Georgia
Contractor: Thompson & Street Company, Atlanta, Georgia

LEHIGH PORTLAND CEMENT CO.

Allentown, Pa.

• LEHIGH EARLY STRENGTH CEMENT • LEHIGH PORTLAND CEMENT
• LEHIGH MORTAR CEMENT • LEHIGH AIR-ENTRAINING CEMENT

ASEE elects officers, conducts conference

The American Society for Engineering Education has announced the election of William T. Alexander, dean of engineering at Northeastern University, Boston, to its presidency for the year beginning in July, 1958.

Other elections of principal officers of the society include: Clarence L. Eckel, dean of engineering at the University of Colorado, re-elected vice-president in charge of regional activities in the western states; and Elmer C. Easton, dean of engineering at Rutgers University, New Brunswick, N. J., named vice-president in charge of activities of the society's instructional divisions.

The 66th annual ASEE meeting including Engineering College Administrative Council and Engineering College Research Council convened June 16-20 at the University of California, Berkeley.

Several ACI members participated in general sessions of the program. C. L. Eckel, University of Colorado, and Glenn Murphy, Iowa State College, presided at two of the breakfast programs; Walter Dreyer, Pacific Gas and Electric Co., took part in a panel discussion regarding secondary education for future engineers.

Gene M. Nordby, National Science Foundation, spoke before the general session on June 18 on "The Role of Government," and J. M. Garrelts, Columbia University, presented a talk on "An Experiment in the Strength of Materials Laboratory" at the general session the following day.

Dean Hollister honored by Lehigh University

Solomon C. Hollister, dean of the college of engineering, Cornell University, Ithaca, N. Y., recently received the honorary doctor of engineering degree from Lehigh University during its 90th commencement exercises.

After varied experience as a practicing engineer following graduation from the University of Wisconsin in 1916, he was appointed professor of structural engineering at Purdue University in 1930 and continued his career as an educator at Cornell University, first as the director of the school of civil engineering and, in 1937, as dean of the college of engineering.

Dean Hollister has freely contributed his time and talents as a technical adviser to many agencies of the federal government and as a member of important defense committees.

Long active in Institute affairs, Dean Hollister is well known in many branches of ACI activity. He is a former director and past president (1932-33) and has been an Institute member since 1917.

Colcrete Structures announces new partnership

A. A. Styner, president, Colcrete Structures, Inc., New York, recently announced a partnership agreement of his firm with Spencer, White and Prentis, New York, foundation and underpinning specialists. The partners will carry out typical prepacked concrete work, dry or underwater, using the Colcrete process.

LOOKING AHEAD

Sept. 15, 1958—6th International Congress on Large Dams, New York, N. Y.

Sept. 21-25, 1958—Prestressed Concrete Institute, Fourth Annual Convention, Edgewater Beach Hotel, Chicago, Ill.

Oct. 13-17, 1958—American Society of Civil Engineers, Annual Convention, Statler Hotel, New York, N. Y.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hotel, Detroit, Mich.

Nov. 18-20, 1958—American Standards Association, Ninth National Conference on Standards, Hotel Roosevelt, New York, N. Y.

Feb. 23-27, 1959—American Concrete Institute, 55th Annual Convention, Statler-Hilton Hotel, Los Angeles, Calif.

ASTM holds 61st annual meeting; concrete and cement studied

The 61st annual meeting of the American Society for Testing Materials was held June 22-27 in Boston. Kenneth B. Woods, Purdue University, was elected president with A. Allan Bates, Portland Cement Association, being elected vice-president. Frank L. LaQue, International Nickel Co., Inc., will continue as senior vice-president.

Paul A. Archibald, Standard Steel Works Division of Baldwin-Lima Hamilton Corp., William L. Fink, Alcoa Research Laboratories, and Harry M. Hancock, Atlantic Refining Co., were elected for 3-year terms on the board of directors.

Others elected to 3-year terms on the ASTM board were: Lawrence A. O'Leary of the W. P. Fuller and Co., and Alfred C. Webber, E. I. du Pont de Nemours and Co., Inc.

In addition to eight technical sessions and 11 symposiums, over 800 committee meetings were held. Symposiums of special interest to the concrete industry were conducted on concrete, cement, durability, and fatigue.

Concrete studied

At the session on concrete, conducted June 23, opening topic of discussion was "Reflectance Tests for Concrete Curing Materials" presented by C. C. Rhodes, M. H. Janson, and M. G. Brown, Michigan State Highway Department. H. L. Cahn and R. V. Mackey, Jr., General Electric Co., spoke on extending concrete highway durability with silicones.

J. J. Waddell, Joseph K. Knoerle and Associates, Inc., presented a paper on testing of prestressing materials and concrete control on the Northern Illinois Toll Highway, followed by Milos Polivka and E. H. Brown, University of California, with a joint discussion on sulfate resistance of portland-pozzolan cement concretes.

At the second concrete session on June 24, G. E. Troxell, J. M. Raphael, and R. E. Davis, University of California, opened the meeting with their joint discussion on "Long-

NEW EDITION!

Completely revised to conform to the recently amended A. C. I. BUILDING CODE

REINFORCED CONCRETE DESIGNS— ALL WORKED OUT!

No more algebraic formulas or calculations to make. Simply locate the table covering the member you are designing, apply span and load requirements, and then *read off directly* concrete dimensions and reinforcing steel data. Follows the latest codes and practices. Send check or money order for your copy, today.

Prepared by The Committee on Engineering Practice

OVER 450 PAGES

$6.00 POSTPAID

10-Day, Money-Back Guarantee
No C.O.D. Orders

CONCRETE REINFORCING STEEL INSTITUTE

38 S. Dearborn St., (Div. J) Chicago 3, Illinois

Time Creep and Shrinkage Tests of Plain and Reinforced Concrete." J. G. Dempsey of Compania Anonima De Concreto followed with a talk based on the effects of fine aggregate characteristics other than grading on strength and durability of concrete.

George Werner, Bureau of Public Roads, also participated in this session, describing the effect of type of capping materials on the compressive strength of concrete cylinders. D. W. Lewis and Fred Hubbard, National Slag Association, presented their joint contribution "Flexural and Compressive Strength Properties of Air-Entrained Concrete with Blast Furnace Slag Aggregate."

Fatigue, durability

At the symposium devoted to the basic mechanisms of fatigue, JoDean Morrow and G. M. Sinclair, University of Illinois, presented a joint paper "Cycle-Dependent Stress Relaxation."

P. V. Johnson, Structural Clay Products Research Foundation, and H. C. Plummer, Structural Clay Products Institute, presented the opening talk at the symposium on durability in structures, basing their discussion on factors affecting durability of structural clay masonry. T. B. Kennedy, Waterways Experiment Station, addressed this session on laboratory testing and the durability of concrete.

Cement

W. C. Hansen, Universal Atlas Cement Corp., opened the cement session June 26 with a discussion on aeration as a cause of false set in portland cement; A. C. Klein and G. E. Troxell, University of California, followed with their joint paper "Studies on Calcium Sulfo-Aluminate Admixtures for Expansive Cements." Stanton Walker and D. L. Bloem, National Ready Mixed Concrete Association and National Sand and Gravel Association, discussed variations in portland cement.

Blackburn joins Bartholomew firm

J. B. Blackburn, formerly associate professor of civil engineering at the University of Maryland, is now affiliated with the firm of Harland Bartholomew and Associates, St. Louis, Mo.

CONCRETE
for
Radiation Shielding

A compilation of seven papers on the use of concrete for shielding nuclear radiation and the calculation of proportions and properties of various heavy concretes.

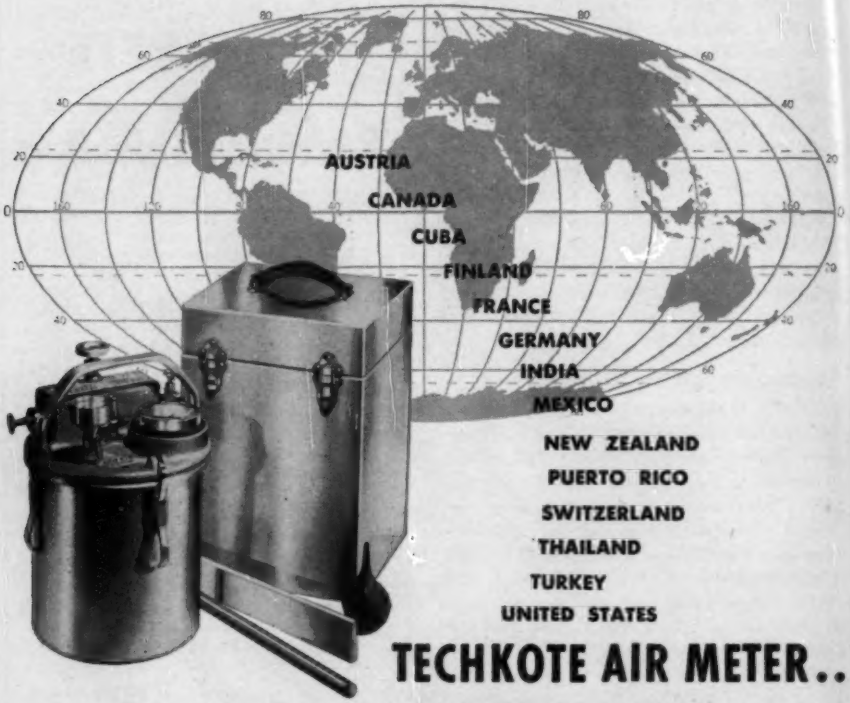
- *Concrete for Radiation Shielding*—Emphasizes factors related to concrete technology and cost, and discusses problems involved in the use of special concrete. Outlines design procedure for concrete shielding.
- *Absorption by Concrete of X-Rays and Gamma Rays*—A discussion of the mechanism of the absorption of x-rays and gamma rays by various shielding materials.
- *Properties of High-Density Concrete Made with Iron Aggregate*—Data on the physical properties of several types of mortar and concrete made with iron-bearing aggregate.
- *Heavy Steel-Aggregate Concrete*—Gives consideration to various mix proportions for heavy concrete and a proportioning procedure for concrete of given strength and density.
- *Properties of Heavy Concrete Made with Barite Aggregates*—Presents data on test results on barite (barium sulfate) in conventional and prepacked concrete where high density is desired.
- *Magnetite Iron Ore Concrete for Nuclear Shielding*—Physical properties and costs of magnetite ore concrete are compared to those of other concretes.
- *Proportioning of Mixes for Steel Coarse Aggregate and Limonite and Magnetite Matrix Heavy Concretes*—A discussion of structural concrete utilizing limonite and magnetite ores as fine aggregate and graded steel scrap as coarse aggregate.

132 pages—offering graphs, tables, and test results

$4.00

($3.00 to ACI Members)

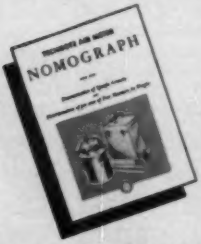
AMERICAN CONCRETE INSTITUTE



TECHKOTE AIR METER..

FIRST CHOICE AROUND THE WORLD

The Techkote Air Meter now provides a multi-range testing instrument that vastly increases the usefulness of the air meter. It is now possible to measure accurately—within a fraction of a degree—entrained air with one instrument on numerous materials such as lightweight concrete, mortar, plaster and soil, in addition to regular concrete.

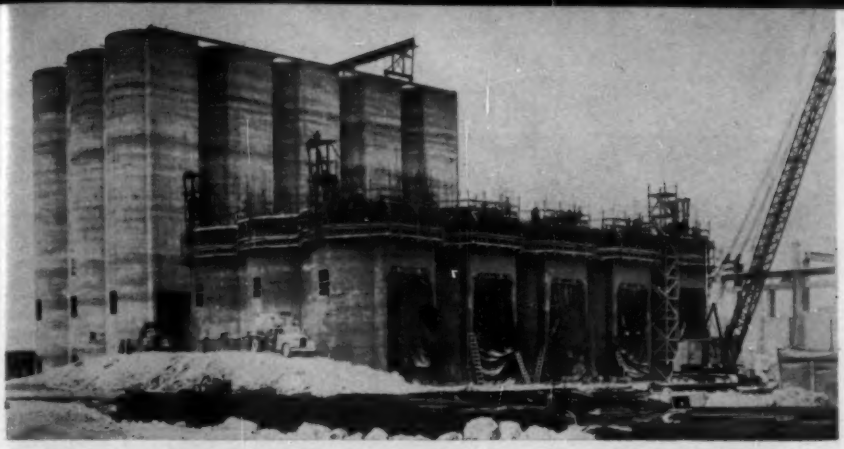


This outstanding new Multi-Range feature, together with the Nomograph and many other exclusives of the Techkote Air Meter, offers a combination unequalled in this type of equipment. TO BE SURE...USE THE FINEST!

TECHKOTE COMPANY

INCORPORATED

600 Lairport Street, El Segundo, California



SILOS FOR CEMENT STORAGE rise at the new multi-million-dollar plant of Lehigh Portland Cement Co. at Miami, Fla., as the second of two banks of silos takes shape. The two banks, containing 15 silos each, will have a total storage capacity of 450,000 bbl of cement. The combined structure is 124 ft high, 225 ft long, and 169 ft wide. Each of the 30 silos is 32 ft in diameter. Five railroad tunnels pass beneath the silos, which are elevated to allow a clearance of 30 ft above the tracks. With this arrangement a full 400-bbl hopper car can be loaded directly from the conical bottom of a silo in about 5 min.

Cement Plant Expansion

CRANES LIFT A 50-TON, 100-ft-long prestressed concrete roof beam into position on the immense raw materials storage building. Beam is one of 39 that will support a roof of 916 precast concrete slabs. Storage building will be 864 x 100 ft, and 88 ft high. Two 17½-ton overhead cranes, with a span of 90 ft, will move back and forth across the entire length of the building, handling raw materials. Plant will have an annual production capacity of 10 million bags of portland cement.

add extra years of life to concrete with
CLINTON WELDED WIRE FABRIC

Experienced contractors know that fabric-reinforced concrete lasts longer. And over the years it looks better and requires much less maintenance than unreinforced concrete. Fabric-reinforcement gives these advantages because it adds the tensile strength of steel to concrete . . . resists the heaving and cracking caused by temperature extremes and moisture content . . . minimizes cracking during setting . . . and distributes stress evenly in all directions.

Even if a crack does develop, the fabric holds it tightly together, preserves the smooth surface, and retards further lengthening of the crack. No other type of reinforcement is so easy to install, yet gives so much extra strength for so little cost.

Clinton Welded Wire Fabric is a top-quality product that is readily available in both East and West, in a wide variety of gages, lengths and widths. On your next job, add the tensile strength of steel with Clinton Welded Wire Fabric, and rest assured that you've built a concrete surface that will last longer with minimum maintenance.

WHEN THEY ASK...
"is it Reinforced"
SAY YES...WITH

CLINTON
WELDED WIRE FABRIC
THE COLORADO FUEL AND IRON CORPORATION

THE COLORADO FUEL AND IRON CORPORATION—Albuquerque • Amarillo • Billings • Boise • Butte • Denver • El Paso • Ft. Worth Houston • Kansas City • Lincoln • Los Angeles • Oakland • Oklahoma City • Phoenix • Portland • Pueblo • Salt Lake City San Francisco • San Leandro • Seattle • Spokane • Wichita • WICKWIRE SPENCER STEEL DIVISION—Atlanta • Boston • Buffalo Chicago • Detroit • New Orleans • New York • Philadelphia • CF&I OFFICES IN CANADA: Montreal • Toronto
CANADIAN REPRESENTATIVES AT: Calgary • Edmonton • Vancouver • Winnipeg 6085

Who's Who This Month

ACI Committee 208

Recently adopted ACI Standard "Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)" appearing on p. 1 was prepared by ACI Committee 208 headed by C. P. Siess, University of Illinois, Urbana. Members of the committee include: the late Arthur P. Clark, formerly research associate for the American Iron and Steel Institute at the National Bureau of Standards, Washington, D. C.; Phil M. Ferguson, University of Texas, Austin; H. J. Gilkey, Iowa State College, Ames; W. H. Jacobs, Rail Steel Bar Association, Chicago; Raymond C. Reese, consulting engineer, Toledo; David Watstein, National Bureau of Standards, Washington, D. C.; A. C. Weber, Laclede Steel Co., St. Louis; and C. A. Willson, American Iron and Steel Institute, New York.

ACI Committee 325

"Recommended Practice for Design of Concrete Pavements (ACI 325-58)" on p. 17 is the work of ACI Committee 325. E. A. Finney, Michigan State Highway Department, Lansing, is committee chairman.

Committee members include: Henry Aaron, Wire Reinforcement Institute, Washington, D. C.; Leo M. Arms, Portland Cement Association, Chicago; John A. Bishop, U. S. Naval Civil Engineering Research and Evaluation Laboratories, Port Hueneme, Calif.; Frank B. Brown, Wire Reinforcement Institute, Washington, D. C.; and Bengt F. Friberg, consulting engineer, St. Louis.

Other members of Committee 325 are: A. T. Goldbeck, National Crushed Stone Association, Washington, D. C.; Robert Horonjeff, University of California, Berkeley; J. D. Lindsay, Illinois Division of Highways, Springfield; Joseph H. Moore, Pennsylvania State College, State College, Pa.; and L. A. Palmer, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.

Completing the 16-man committee are: G. S. Paxson, Oregon State Highway Department, Salem; Thomas B. Pringle, Office, Chief of Engineers, Department of the Army, Washington, D. C.; L. W. Teller, Bureau of Public Roads, Washington, D. C.; William Van Breemen, New Jersey State Highway Department, Trenton; Kenneth B. Woods, Purdue University, West Lafayette, Ind.; and F. N. Wray, Highway Research Board, Washington, D. C.

ACI Committee 617

The revised standard "Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)" appearing on p. 53 was developed by ACI Committee 617 under the chairmanship of H. F. Clemmer, Government of the District of Columbia, Washington D. C., and supersedes the 1951 ACI standard on the same subject.

Members of the committee are: E. A. Abdun-Nur, consulting engineer, Denver; Harold Allen, Bureau of Public Roads, Washington, D. C.; A. A. Anderson, Portland Cement Association, Chicago; and E. W. Bauman, National Slag Association, Washington, D. C.

Also serving on the committee are: John A. Bishop, U. S. Naval Civil Engineering Research and Evaluation Laboratories, Port Hueneme, Calif.; Frank B. Brown, Wire Reinforcement Institute, Washington, D. C.; A. T. Goldbeck, National Crushed Stone Association, Washington, D. C.; W. E. Hawkins, North Carolina State Highway Department, Raleigh; and T. E. Howell, Caterpillar Tractor Co., Peoria, Ill.

Balance of the committee includes: Fred Hubbard, National Slag Association, Youngstown, Ohio; Theodore J. Kauer, The Holmes Construction Co., Wooster, Ohio; Phillip L. Melville, Office, Chief of Engineers, Department of the Army, Washington, D. C.; Bailey Tremper, California Division of Highways, Sacramento; and Stanton Walker, National Ready Mixed Concrete Association and National Sand and Gravel Association, Washington, D. C.

ACI Committee 711

"Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)" on p. 83 was prepared by ACI Committee 711 with Leo M. Legatski, University of Michigan, Ann Arbor, serving as chairman. It supersedes ACI 711-53.

Ralph W. Adams, consulting engineer, Ypsilanti, Mich.; J. H. Appleton, Alabama Cement Tile Co., Birmingham, Ala.; Fred C. Bamman, F. C. Bamman Precast Concrete Corp., Hallandale, Fla.; and Merwin G. Beavers, Texas Concrete Products Corp., Austin, are members of this committee.

Other members of Committee 711 are: Frank Bromilow, New Mexico College of Agriculture and Mechanic Arts, State College, N. M.; S. J. Chamberlin, Iowa State College, Ames; William C. Green, Texcrete Structural Products Co., Dallas; Harold B. Hemb, Concrete Products, Inc., Skokie, Ill.; and George E. Large, Ohio State University, Columbus.

Remaining members of the committee are: F. N. Menefee, University of Michigan, Ann Arbor; C. F. Moore, Hutton and Hutton, Hammond, Ind.; O. Neil Olson, Marquette University, Milwaukee; Gayle B. Price, Price Brothers Co., Dayton; Robert A. Rossi, Libbey-Owens Ford Glass Co., Toledo; and R. N. Vakil, University of Wisconsin, Madison.

Richard C. Mielenz, Vladimir E. Wolkodoff, James E. Backstrom, and Harry L. Flack

Four authors have cooperated in the preparation of Part 1 of a four-part paper entitled "Origin, Evolution, and Effects of the Air Voids System in Concrete." Part 1, "Entrained Air in Unhardened Concrete," appears on p. 95 of this month's JOURNAL.

Richard C. Mielenz, director of research, The Master Builders Co., Cleveland, active in ACI committee work and a frequent contributor of technical papers for JOURNAL publication, has been an Institute member since 1945. He received his PhD degree in 1939 in geological sciences at the University of California, and began his career as a geologist in 1939 with the Standard Oil Co. of California. In 1941 he joined the Bureau of Reclamation where he remained until 1956 when he made his present affiliation. His main professional activities have been related to selection and testing of aggregates, the microscopy of concrete, concrete admixtures, and the petrography and engineering performance of rocks and soils.

Dr. Mielenz received the ACI Wason Medal for noteworthy research in 1948 for the paper "Chemical Test for Reactivity of Aggregates with Cement Alkalies; Chemical Processes in Cement-Aggregate Reaction," which he co-authored with K. T. Greene and E. J. Benton (ACI JOURNAL, Nov. 1947, *Proc.* V. 44, pp. 193-224). He received the Sanford E. Thompson Award of the American Society for Testing Materials in 1949, 1951, and 1954.

Currently chairman of ACI Committee 116, Nomenclature, Dr. Mielenz is a member of ACI Committee 212, Admixtures.

Vladimir E. Wolkodoff, senior engineer, Carborundum Co., Niagara Falls, N. Y., was formerly petrographer with the Bureau of Reclamation. Harry L. Flack is an engineer with the Bureau of Reclamation in Denver.

James E. Backstrom graduated from Mississippi State College in 1933 and then joined the Bureau of Reclamation where he was assigned to the dam design section. In

ACI *Proceedings* in microfilm

ACI members and JOURNAL subscribers may obtain complete *Proceedings* volumes (including discussion) in microfilm. This is an economical means of maintaining a file of ACI material with the minimum storage space. *Proceedings* V. 46 (Sept. 1949 to June 1950) through *Proceedings* V. 53 (July 1956 to June 1957) are available.

Proceedings for the last two volume years (V. 52 and V. 53) are available at $4.25 and $5.65 respectively. Volume 51 costs $4.40. For additional price information and orders, write University Microfilms, 313 N. First St., Ann Arbor, Mich.

MARACON

MARACON

WATER-REDUCING ADMIXTURES FOR CONCRETE

MARACON promotes more complete hydration of cement particles and permits a substantial reduction in the unit water content without loss of plasticity or consistency of the mix. Low water content means . . .

HIGHER STRENGTH

AND BETTER CONCRETE AT LOWER COST

Concrete containing Maracon being poured by Heiser Ready-Mix Company of Wausau, Wis.

With Maracon you can:

1. LOWER CONCRETE COSTS: —
   - A. Maintain slump and workability at low W/C ratios.
   - B. Attain higher strengths without increasing cement content of a mix.
   - C. Permit economical redesign of conventional concrete mixes.
2. IMPROVE CONCRETE QUALITY: —
   - A. Minimize shrinkage in concrete before and after hardening, due to lower water content and more complete hydration of cement.
   - B. Decrease permeability.
   - C. Achieve greater density and higher durability factors.

MARACON also reduces water requirements in concrete mixes containing pozzolanic materials.

DISTRIBUTORS

West of the Rockies, Hawaii and Alaska
Admixtures, Inc.
965 N. Fair Oaks Ave.
Pasadena, California

East of the Rockies
Truscon Laboratories
Division of
Devoe & Raynolds Co., Inc.
1700 Caniff, Detroit 11, Mich.

(Mississippi only)
Ready-Mix Concrete Company
Meridian, Mississippi

MARATHON
A Division of American Can Company
CHEMICAL SALES DEPARTMENT
ROTHSCHILD, WISCONSIN

Use this coupon for more information.

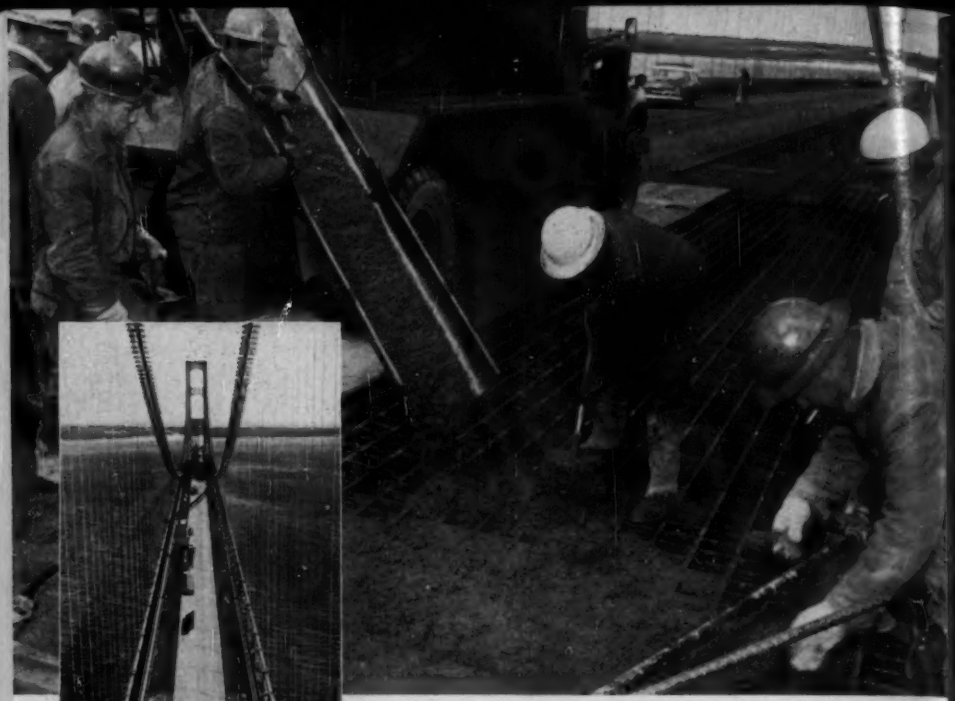
MARATHON • CHEMICAL SALES DEPT.
ROTHSCHILD, WIS.
A division of American Can Co.
Send additional information on Maracon to: —

NAME:_____

COMPANY:_____

ADDRESS:_____

J-78



Mackinac Bridge, Michigan

Owner: Mackinac Bridge Authority

Consulting Engineer: Dr. D. B. Steinman

Associate Consultant: Glenn B. Woodruff

Substructure Contractor: Merritt-Chapman & Scott Corp.

Superstructure Contractor: American Bridge Div. U.S. Steel Corp.

POZZOLITH Ready-Mixed Concrete: Louis Garavaglia Co.

Mackinac Bridge... POZZOLITH* Employed in Lightweight Concrete in the Deck to Improve Workability and to Reduce Shrinkage

*** POZZOLITH — Master Builders trademark for its time-tested water-reducing, air-entraining admixture for concrete.**

THE MASTER BUILDERS CO.
DIVISION OF AMERICAN-MARIETTA CO

General Offices: Cleveland 3, Ohio • Toronto 9, Ontario • *Export:* New York 17, N. Y.
Branch Offices In All Principal Cities • Cable: Mastmethod, N. Y.

1935 he transferred to the bureau's concrete laboratory where he engaged in cement, pozzolan, and concrete investigations for Grand Coulee, Shasta, and Friant dams.

From 1939 to 1942 he was employed in the concrete design section of the Tulsa District, U. S. Army Corps of Engineers, and later on the construction of Canton Dam in Oklahoma. In 1942 he entered the Civil Engineer Corps of the U. S. Navy and served in various capacities in this country and in the South Pacific area until 1946.

Mr. Backstrom then returned to the Bureau of Reclamation's materials laboratory where he has been engaged primarily in concrete research. He presently is head of the materials, mixes, and durability section of the Concrete Laboratory Branch.

He is a member of ACI, the Highway Research Board, and serves on Committee B-1, Durability of Concrete—Physical Aspects, of the Highway Research Board.

Donald D. Meisel, Cyril D. Jensen, and Walter H. Wheeler

"Load Test on Flat Slab Floor with Embedded Steel Grillage Caps," p. 123, was submitted by Donald D. Meisel, Cyril D. Jensen, and Walter H. Wheeler.

Donald D. Meisel, chief engineer, E. L. Conwell and Co., Philadelphia, graduated from Pennsylvania State University in 1949. He taught in the fields of materials testing and concrete in the civil engineering department at Bucknell University, Lewisburg, Pa., for several years. He has been in his present position for the past 6 years, specializing in concrete consultation, materials testing, inspection of concrete and other construction materials. Mr. Meisel is a member of ACI and a number of other professional societies.

Cyril D. Jensen, professor of civil engineering, Lehigh University, Bethlehem, Pa., graduated from the University of Minnesota. He has been active in structural research (mostly on welded steel) since 1927, his investigations being conducted in the Fritz Engineering Laboratory at Lehigh University.

During World War II he developed underwater cutting and welding for the Navy at the U. S. Naval Engineering Experiment Station, Annapolis, Md. Since then he has directed research on bomb damage analysis, bomb shelters, investigation of causes of faulting in concrete pavements, and welded interior beam-column connections. Another assignment since the war has been the development of soil mechanics at Lehigh University. In addition to his teaching duties, he is graduate officer in the department of civil engineering.

Walter H. Wheeler, with 50 years of private practice first as engineer and contractor and then as consulting engineer, is still actively engaged in his profession. At this time his projects are widely scattered from Long Island to Houston to Hawaii, with highway and railroad bridges in two states on his work schedule.

Graduating from the University of Minnesota in 1906, Mr. Wheeler started his career with Hones and Wheeler, Denver. For Colorado Fuel and Iron Co. he designed and built successful reinforced concrete battery walls for beehive coke ovens (temperature on back about 1000 F) used until process was abandoned about 6 years ago.

A long and impressive list engineering accomplishments followed: the Fort Snelling Mendota Bridge, Hennepin County, Minn., said to be the largest multiple rib arch bridge of reinforced concrete in the world; U. S. Appraiser's Stores, Baltimore, Md., using flat slab supported on steel columns and steel grillages embedded in slab to replace flared concrete column capitals; continuous girder three-span reinforced concrete bridge in which the two river piers are hinged at the bottom; YMCA in Honolulu. A 2,000,000-bushel grain elevator in St. Paul used steel pipe columns and flat slabs with steel grillages supporting bottoms of interstice bins and ring girders in main bins. He is inventor of "Smooth Ceilings" system flat slab construction used in many buildings throughout United States, Canada, and elsewhere.

Mr. Wheeler received the outstanding achievement award gold medal and citation for professional achievements from University of Minnesota in 1952. He has been a member of ACI since 1935 and is currently a member of ACI Committee 318, Standard Building Code.

I. D. MacKenzie

"Heavy Media Processing of Gravels in New Brunswick" appearing on p. 133 was

prepared by I. D. MacKenzie, geologist, Shawinigan Engineering Co., Ltd., Montreal.

Mr. MacKenzie graduated from Queen's University in 1940 with a degree in geology and mineralogy and joined the Shawinigan staff shortly thereafter.

His experience has included direct supervision of concrete repair programs on large concrete dams, bulkheads, and powerhouse intakes. At the present he is engaged with development and supervision of concrete repair programs, supervising inspection of existing concrete structures, and drawing up recommendations for repair as well as supervision of mix proportioning and concrete control on the construction of hydroelectric developments and other concrete structures.

LOS ANGELES IN '59

ACI's Technical Activities Committee is already looking forward to the 1959 convention in Los Angeles; program plans are taking shape.

Those who intend to offer papers for presentation at the 55th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1958, and final manuscripts of accepted papers will be due Jan. 1, 1959.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1959 program.

Fazlur R. Khan and Andrew J. Brown

The large number of girders required for the Air Force Academy bridges in Colorado Springs, and their importance as major structures, made it advisable and possible to conduct the "Load Test of 120-Ft Precast, Prestressed Bridge Girder" (p. 139) reported by Fazlur R. Khan, civil engineer, Dacca, Pakistan, and Andrew J. Brown, chief structural engineer, Skidmore, Owings, and Merrill, Chicago.

Fazlur R. Khan graduated from the University of Dacca in Pakistan. He was active there in designing reinforced concrete structures including bridges and also taught reinforced concrete design for undergraduates.

In 1955 he received his PhD in civil engineering from the University of Illinois. During his stay at the university he spent some of his time in concrete research as a research assistant. Before returning to Pakistan, he was senior structural designer for Skidmore, Owings, and Merrill.

Andrew J. Brown graduated from Technical University of Budapest in 1919 and came to the United States in 1923. Much of his experience has been in reinforced concrete design and construction. Between the years 1930 to 1936, working for Strauss and Paine, Inc., he was in charge of the design of the piers, anchorages, anchor pylons and the Presidio approach structure of the Golden Gate Bridge. From 1939 to 1941 as structural designer with the Department of Subways and Superhighways, City of Chicago, he designed much of the subway tunnel system.

In 1945 he joined Skidmore, Owings and Merrill's Chicago office in his present capacity. Here he has designed or was in charge of design of various types of concrete structures, such as: Medical Center of Ohio State University; New York Life Insurance Co.'s Lake Meadows development in Chicago; lift slab construction for an Antioch College dormitory; and more recently, the foundation structures, retaining walls, and bridges of the Air Force Academy project at Colorado Springs.

Gabriel Scott Brown

Gabriel Scott Brown, former president and chairman of the board of Alpha Portland Cement Co., Easton, Pa., died recently.

A graduate civil engineer, Mr. Brown joined Alpha in 1898 as chief clerk at the Alpha, N. J., plant. He was elected secretary-treasurer the following year and was named to the board of directors in 1906. In 1914 he was elected president and served until 1935 when he was made chairman of the board. He became honorary chairman in 1949.

Mr. Brown was a graduate of Hillman Academy, Wilkes Barre, and was president of the Princeton University class of 1894.

Collings and Vranich form consulting firm

Two ACI members have announced the formation of the consulting engineering firm of Collings-Vranich and Associates with headquarters in Milwaukee. The principals of the firm are William T. Collings and Emil F. Vranich. Mr. Collings was project engineer for Klug and Smith Co. of Milwaukee prior to his present affiliation, and Mr. Vranich was vice-president of the consulting firm of Robert J. Strass, Inc.

Air found important to mortar durability

A correction should be made in "ASTM Conducts Committee Week, Mortar Symposium" in the ACI News Letter, May 1958, p. 29. Research by W. L. Zemaitis at Lafayette College, Easton, Pa., indicates that air content has a highly significant effect on the durability of mortar. Different limes used to prepare mortar result in different air contents and water demands, thus also influencing durability as well as compressive strength, expansion, and shrinkage. The News Letter story should not have stated that "lime is the most important constituent . . . "

the beginning of BIG trouble

STOP IT NOW...

RESTORE CONCRETE SURFACES

with SHOTCRETE*

Spalled concrete generally results from oxidized reinforcing rods. Unless decay is halted, serious structural weakness and huge repair bills will result.

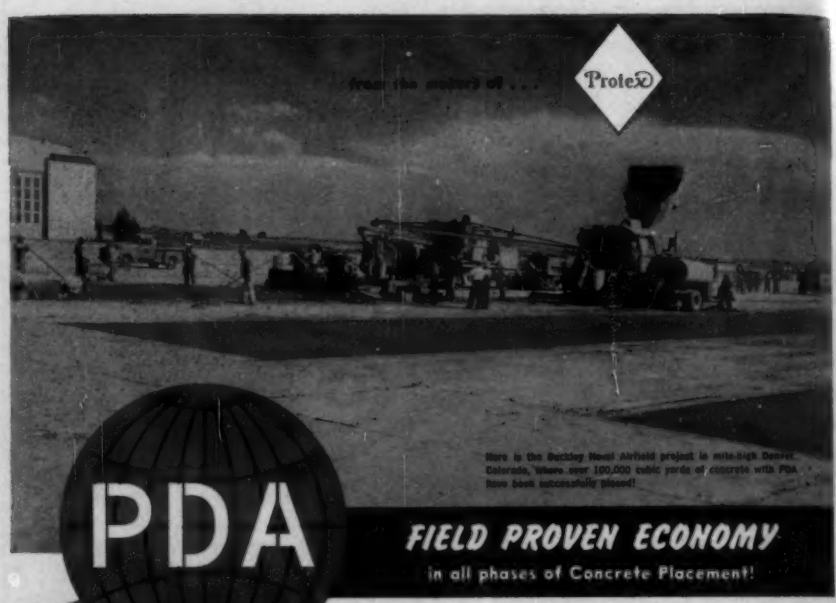
With shotcrete, skilled Western technicians can restore disintegrated concrete structures to original soundness and strength. All work done under contract, fully insured and guaranteed. No materials for sale.

* Universally accepted term for pneumatically-placed mortar.

other Western services

- TUCKPOINTING
- BUILDING CLEANING
- SUB-SURFACE WATER PROTECTION

WESTERN WATERPROOFING CO., Inc.
A Missouri Corporation and its affiliation
RESTORATION COMPANY, INC.
Engineers and Contractors • 1723 Syndicate Trust Bldg. • St. Louis 1, Mo.
NATIONWIDE SERVICE


from the makers of . . .
Protex
Here is the Buckley Naval Airfield project in mile-high Denver, Colorado, where over 100,000 cubic yards of concrete with PDA have been successfully placed!

PDA

FIELD PROVEN ECONOMY
in all phases of Concrete Placement!

PROTEX Dispersing Agent . . .

PDA increases strength at any age through water reduction and cement dispersion, improving all the desirable characteristics of concrete, giving a "live" concrete with protection against "hot weather" slump loss and segregation—allows the placing of controlled durable air entrained concrete without plastic shrinkage cracks. PDA is a selective initial retarder—retarding only the initial set of concrete for extended vibration and finishing time (not delaying form stripping time) YET it gives no retardation in winter concreting — thus year around benefits are obtained with "all seasons" PDA.

PDA . . . packaged in a durable bag . . . comes to you in convenient powder form . . . easily mixed and dispensed . . . complete dispensing installations available upon request.

This is the Bread Plant of the Indiana-Michigan Electric Company at Sullivan, Indiana, where over 55,000 cu yds of concrete with PDA are being successfully placed.

Proven, Dependable, Adaptable

Proven in the field . . . PDA insures the successful and economical placement of better quality concrete!

Dependable—YES! Proof positive from Government and private projects. Also backed by the world-known and world-respected PROTEX name!

Adaptable to any need . . . PDA improves any Pre-stressed, Slip-form, Light-weight, Tunnel-lining, Tilt-up or Lift-slab concrete project!

PDA's basic material is purified and desugared having been field tested and proven over past years giving you all the well known benefits — greatest water reduction — cement dispersion — maximum workability and durability — plus PROTEX economy and reliability.

FOR BETTER AND MORE ECONOMICAL PLACEMENT OF CONCRETE, SPECIFY AND USE PDA . . . FROM THE MAKERS OF PROTEX!

Please send new, informative FREE booklet "PDA - Protex Dispersing Agent"

Firm Name_____

Attention of_____

Address_____

City_____ State_____

Protex

AUTOLENE LUBRICANTS COMPANY

• Industrial & Research Division

1331 WEST EVANS AVENUE DENVER 9, COLORADO

® T.M. Reg.

Honor Roll

January 1—June 30, 1958

You, as a member of ACI, can further the influence of the Institute and its work. Through your efforts, others throughout the world are able to have access to information in furthering concrete and concrete construction. This effort on your part is depicted each month by the Honor Roll. Is your name on this list?

Blas Lamberti....14
Joseph J. Waddell....11
Phil M. Ferguson.... 8
Ernest Maag.... 6
Jerome M. Raphael.... 5
C. P. Siess.... 5
Ernest L. Spencer.... 5
J. P. Thompson.... 5
Moises Bendahan.... 4
Celso A. Carbonell.... 4
Carl H. Koontz.... 4
L. M. Legatski.... 4
Frank E. Legg, Jr.... 4
James A. McCarthy.... 4
Clarence A. Walkwitz.... 4
Jaime de las Casas.... 3
P. E. Ellen.... 3
Martin J. Gutzwiller.... 3
Paul A. Hansen.... 3
Samuel Hobbs.... 3
Kenneth M. Huber.... 3
Daniel S. Ling.... 3
H. P. Mittet.... 3
H. C. Pfannkuche.... 3
E. Schmid.... 3
Byron P. Weintz.... 3
William M. Avery.... 2
Nicandro Jose Barboza.... 2
Luther E. Bell.... 2
O. R. Bell.... 2
M. R. Berretti.... 2
Vincent R. Cartelli.... 2
Miles N. Clair.... 2
W. L. Collier.... 2
W. S. Cottingham.... 2
Aleck S. Evans.... 2
Arch Hull.... 2
Thomas C. Kavanagh.... 2
F. R. Khan.... 2
R. G. King.... 2
Carl O. Knop.... 2
E. Vernon Konkel.... 2
George Kurio.... 2
James R. Libby.... 2
Ciceron Hiedra Lopez.... 2
William McGuire.... 2
Douglas McHenry.... 2
George A. Mansfield.... 2
Bryant Mather.... 2
Frank B. May.... 2
James A. Murray.... 2
Wendell H. Nedderman.... 2
Walter H. Price.... 2
Abdur Rahman S. Rasul.... 2
Clarence Rawhauser.... 2
Francesco Sardella.... 2
C. H. Scholer.... 2
George Shervington.... 2
Howard Simpson.... 2
Donald L. Strange—Boston.... 2
T. W. Thomas.... 2
Ellis S. Vieser.... 2
Jose Antonio Vila.... 2
Ivan A. Villamil.... 2
Tsu-ming Yang.... 2

New Members

The Board of Direction approved 67 Individual applications, 21 Juniors, and 12 Students making a total of 100 new members. Considering losses due to deaths, resignations, and nonpayment of dues the total membership on June 1, 1958, was 9483.

Individual

ALINN, ALPHONSE J., Cleveland, Ohio (Engr., Supv. of Constr., Sam W. Emerson Co.)

ALLEN, A. D., JR., Jacksonville, Fla. (Gen. Mgr., H. J. High Constr. Co.)

BARBIERO, VICTOR, Westfield, N. J. (Sec. & Chf. Engr., Arthur Venneri Co.)

BILLINGS, DR. ORMAN B., New Brunswick, N. J. (Asst. Dir., Special Prod. Div., Johnson & Johnson)

BIZZARRI, ETTORE CARDASSI, Caracas, Venezuela (C. E.)

BRENNAN, JOHN J., New York, N. Y. (Vice-Pres., in charge of estimating & sales, Brennan & Sloan, Inc.)

BROWN, ALEXANDER, Georgetown, British Guiana, S. A. (Constr. Supt., British Guiana Gov't Public Bldgs.)

CHOO, WOO WON, Seoul, Korea (Chf. Engr., Highway Section, Bur. of Pub. Wks., Ministry of Home Affairs)

CRAIL, HOWARD E., Florence, Ky. (Asst. Engr., Design & Supv. New Constr., Cincinnati Gas & Electric Co.)

CROSSING, L. B., Whitedog Falls, Ont., Canada (Asst. Div. Engr., Ontario Hydro-Electric Power Comm.)

DAVIDSON, EDWARD W., Seattle, Wash. (Struct. Engr., Young, Richardson & Carleton)

DICKINSON, GEORGE G., El Paso, Texas (Mgr. & Co-Owner, Texas Testing Labs., Inc.)

DUBASH, RUTON J., Bombay, India (Cons. Engr. & R. C. C. Specialist)

DURKIN, FRANCIS XAVIER, Valley Stream, N. Y. (Struct. Designer, Burns & Roe, Inc.)

ENRICO, TONY, Morenci, Arizona (Struct., Mech. & Gen. Design Engr., Phelps Dodge Corp.)

FEIGENSPAN, DENNIS E., San Antonio, Texas (Cons. Engr.)

GINNER, CLIVE E., Oakland, Calif. (Proj. Engr., Kaiser Aluminum & Chemical Corp.)

GROBB, FREDERICK S., Brantford, Ont., Canada (Dir. of Tech. Services, G. F. Sterne & Sons)

HALL, KEN, Cloverdale, B. C., Canada (Small Mfr. of tile & other terrazzo products)

HANSEN, IB S., Washington, D. C. (Assoc., Mgr. of Struct. Design Office, Eramian, Anthony & Hosley)

HEER, JOHN E., JR., Louisville, Ky. (Assoc. Prof. of Civil Engrg., Univ. of Louisville)

HOWELL, ROBERT, Valley Stream, L. I., New York (Struct. Designer, Bechtel Assocs.)

HUHN, GUSTAVE W., East Orange, N. J. (Constr. Repres., Public Housing Admin.)

HUNNICUTT, JAMES MADISON, Birmingham, Ala. (Struct. Engr. & Designer, Menefee & Smith)

JANET, DAVID K., Salvador, Bahia, Brasil (Supv. Field Engr., Kellogg Pan American Corp.)
JESSUP, GEORGE P., SR., Newfield, N. Y. (Proj. Engr., Cornell Univ.)
JOHNSON, MARTIN A., New York, N. Y. (Area Engr., "Constr.," Greenland Contrs.)
JONES, JACK H., Toronto, Ont., Canada (Chf. Engr., The Toronto Harbour Comms.)
KAM, MERLE T., Los Angeles, Calif. (Struct. Designer, Holmes & Narver)
KIRK, DON W., Ft. Worth, Texas (Cons. Struct. Engr.)
KISSELL, DONALD D., Branford, Conn. (Struct. Engr., Seelye, Stevenson, Value & Knecht)
KLITZKE, RICHARD L., W. Lafayette, Ind. (Grad. Stud., Purdue Univ.)
KRAFT, RAY, Tulsa, Okla. (Struct. Advisor, The Refinery Engrg. Co.)
KRAUS, DR. FELIX M., Montreal, Canada (Cons. Struct. & C. E.)
KYNAST, FRANK W., New York, N. Y. (Paving Engr., Resident Repres., M. C. Tomasetti)
LEE, THOMAS B., Nashville, Tenn. (Asst. City Engr., City of Nashville)
LIRA, ALFONSO DELGADO, Arequipa, Peru (C. E., Contr., Bldg. Constr.)
LOEDOLFF, GERRIT FRANCOIS, Stellenbosch, South Africa (Sr. Lecturer in C. E., Univ. of Stellenbosch)
MATHIS, SAM J., Caracas, Venezuela (Head, Gen. Engrg., Creole Petroleum Corp.)
MCALLISTER, ROBERT J., Pittsburgh, Pa. (Struct. Engr., Duquesne Light Co.)
MILLER, LAWRENCE C., Spokane, Washington (Field Engr., PCA)
MOLEIRO HIJO, RODOLFO, Caracas, Venezuela (C. E.)
MORTON, J. ROBERT, Syracuse, N. Y. (Asst. to Pres., New Prod. Research, Vega Industries, Inc.)
NEAL, JACK, Des Plaines, Ill. (Constr. Engr., Joseph K. Knoerle & Assocs., Inc.)
NORTON, JOHN C., Traverse City, Mich. (Cons. Engr., Partner, Norton, Kobbins & Assoc.)
OESAU, HANS, Chicago, Ill. (Engrg. Draftsman, City of Chicago, Bureau of Engrg.)
OYSTER, GEORGE W., Green Bay, Wis. (Asst. Design Engr., Wis. Public Service Corp.)
PAINE, E. E., Arvida, Que., Canada (Mgr., Engr., Arvida Mix & Supply Co., Ltd.)
PANARESE, WILLIAM C., Chicago, Ill. (Rlwy. Repres. for Struct. & Rlwys. Bur., PCA)
PARSONS, JOHN K., Culver City, Calif. (Struct. Design Engr., Morris K. Goldsmith)
PEREZ DEL CAMPO, ARMANDO, Banes, Ote, Cuba (Asst. Engr., United Fruit Sugar Co., Banes Div.)
PFUTZENREUTER, ORRIS O., Duluth, Minn. (Field Engr., PCA)
PROUT, W. J., St. Marys, Ont., Canada (Chem., St. Marys Cement Co., Ltd.)
ROBERTS, JACK, Hammond, Ind. (Struct. Cons., Concrete & Steel)
ROCHFORT, GRAEME G., Cabramurra, NSW, Australia (Engr., Assists in Mtls. Control, Snowy Mts. Hydro Elect. Auth.)
RODRIGO M., RAFAEL, Mexico D. F., Mexico (C. E., Depto del D. F. of Edificios)
SELLERI, LUIGI MELA, Caracas, Venezuela (C. E.)
SHELL, RICHARD G., Dayton, Ohio (Struct. Engr., Yount, Sullivan & Lecklider)
SHIRAYAMA, KAZUHISA, Tokyo, Japan (Research Member, Bldg. Research Inst., Ministry of Constr., Japanese Gov't)
SMITH, HARRY G., Pittsburgh, Pa. (Engrg. Mgr., Auburn & Assocs.)
SNAPP, VICTOR E., Houston, Texas (Sales Engr., Texas Lightweight Aggregate Co.)
STINTON, F. M., Kansas City, Mo. (Mgr., Preload Midwest Constr. Co.)
TAEGE, ROBERT L., Chicago, Ill. (Mgr. Comm. Constr., Gen. Supt., The Edward Gray Corp.)
THOMAS, WALLACE A., St. Louis, Mo. (C. E., Res. Engr. St. Louis Water Dept.)
TRITSCHLER, ENRIQUE FLORES, Buckinghamshire, England (C. E., Research work on prestressed concrete, Federation of British industries)
UYANIK, M. E., Raleigh, N. C. (Prof., N. C. State College)
WIGHT, GEORGE, Barrington, Ill. (C. E., Hwys., Sewer & Water, Wight & Co.)

Junior

BRYANT, J. STEPHEN, Panama City, Fla. (Proj. Engr. S. A. Bryant & Assoc.)
COO, JOHN UY, Chicago, Ill. (Civil Engrg., Hwy. Dept., State of Ill., Dept. of Pub. Wks. & Bldgs. Div.)
COX, DAVID GORDON, Christchurch, New Zealand (Research Stud., Engrg. School, Canterbury Univ.)
DALBERG, ROBERT E., Struthers, Ohio (Detail Draftsman in Design Engrg., U. S. Steel Corp., Youngstown Dist. Wks.)
DINIZ, JOSE Z. F., Gainesville, Fla. (C. E., Univ. of Fla.)
GARCIA V., ALFONSO, Monterrey, N. L., Mexico (Constr. Co.)
GLEASON, MICHAEL N., Gilmore City, Iowa (U. S. Marine Corps)
HAAS, JOSEPH W., Quincy, Ill. (Proj. Engr. for Hadley Creek Watershed Protection Proj., Soil Conservation Service, U. S. Dept. of Agric.)
HRIBAR, JOHN P., Euclid, Ohio (Designer, Detailer, Howard, Needles, Tammen & Bergendoff)
MACARIO, ANGEL RAFAEL, Caracas, Venezuela (2nd Chf. for Engrg. Dept., Petrogas C. A.)
MASTIN, BOB, Santa Fe Springs, Calif. (Chf. Draftsman & Checker, Rockwin Prestressed Concrete Corp.)
REEMSNYDER, HAROLD SCOTT, Pittsburgh, Pa. (Struct. Designer, Richardson, Gordon & Assocs.)
REINHARDT, JOHN JOSEPH, Kanab, Utah (C. E., Insp., Design, Est., USBR)
ROBERTS, ASHBY R., Grants Pass, Oregon (Struct. Insp., Bridge Dept., Oregon State Hwy. Dept.)
RODRIGUEZ, JAIME RODRIGUEZ, Bogota, Colombia (Ingeniero Calculista del Depto. de Central, Secretaria de Obras Publicas Bogota)
SADDE, ALEJANDRO OSCAR, Caracas, Venezuela (C. E., Sadde Bros. Co.)
SAMMAN, WILLIAM, Kuwait, Arabian Gulf (Dist. Engr., Pub. Wks. Dept.)
SANTOS, JOSE ANGEL, Monterrey, N. L., Mexico (Grad. Asst., Lehigh Univ.)
STODOLA, PAUL R., Oroville, Calif. (Jr. C. E., Field Concrete Lab., State of Calif., Dept. of Water Resources & Div. of Design & Constr.)
WARNE, JAMES M., Edmonton, Alta., Canada (Engr., Alta. Dept. of Pub. Wks.)
WILLIAMS, ALAN, Salisbury, Southern Rhodesia (Engr., Div. of Roads & Road Traffic)

Student

CARIGNAN, NEIL M., Oxford, Mass. (Worcester Polytechnic Inst.)
COFFEY, CARLETON TIMOTHY, JR., Berkeley, Calif. (Univ. of Calif.)
DALTON, PAUL M., Worcester, Mass. (Worcester Polytechnic Inst.)
HARDT, WILLIAM F., Bloomfield, N. J. (Cornell Univ.)
MOON, DANIEL, Yardley, Pa. (Alfred State Tech.)
NARANJO CARRILLO, JOSE, Cabimas, Zulia, Venezuela (Univ. Nacional del Zulia)
ROLAND, OLIVE M., Detroit, Mich. (Wayne State Univ.)
SHAH, LAXMIDAS DAHYABHAI, South Bend, Ind. (Univ. of Notre Dame)
THOMAS, MARVIN S., Ann Arbor, Mich. (U of M)
TOMA, DAN N., Dearborn, Mich. (Wayne State Univ.)
VALLARINO, EDUARDO, JR., Baton Rouge, La. (L. S. U.)
WOOD, LEWIS K., Berkeley, Calif. (Univ. of Calif.)

Erratum

The following correction should be made in "Precast Reinforced Concrete Slab Bridges with Stiffened Edges," by Andrew Gallis, which appeared in the June JOURNAL.

p. 1090—Under "bending moments and stresses in center of curb in longitudinal direction," f_c should be 700 psi, not 760 psi.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute
P. O. Box 4754, Redford Station
Detroit 19, Michigan

Date_____

Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies)	$15.00
Individual Members (All other foreign countries)	12.00
Corporation Members	50.00
Contributing Members	100.00
Junior Members—nonvoting (under 28)	7.50
Student Members—nonvoting (under 28)	5.00

(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for_____membership
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency or government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

Sec. 4. All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversay of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 6. Applications for and resignations from membership and requests for change of representatives of Corporation or Contributing Members shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

(cut here)

Name _____
Please Print

Date of Birth _____
Month Day Year

Title or Position _____

Name of Firm or Organization _____

☐ **Business Address** _____

☐ **Resident Address** _____
(Please check address to which you wish mail and publications sent)

Nature of Firm's Business _____

The ACI Membership Directory will be sent—as available—only on request. Check here if you wish to receive the latest edition. ☐

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Strand chuck for prestressing

A tool, said to bypass common difficulties encountered in existing gripping devices for wire rope, has jaws made of chrome molybdenum alloy steel with precision

cut thread. The jaws are held together with a simple spring clip and are held in their gripping position by a rugged wire spring attached to the chuck cap (see photo).

Workman slides the chuck onto prestressing strand, moves it to desired position, and jaws automatically assume proper grip. The manufacturer reports that chuck is designed to withstand all tensions encountered in prestressing. The jaws, marked with the size of strand they are intended to grip, do not extend out of the chuck when in use.—Supreme Products Corp., 2222 South Calumet Ave., Chicago, Ill.

Dispersing agent used at airfield

A concrete water reducing agent, PDA (Protex Dispersing Agent), reportedly gives maximum water reduction, higher strengths, and better cement dispersion in all types of mixes.

More than 100,000 cu yd of concrete were recently placed on a U. S. Naval Airfield, Denver, using 6 sacks of cement per cu yd, and ¼ lb PDA per sack. This admixture is said to have permitted a 20 percent reduction in the unit water content, and gave flexural strength greater than the required 600 psi at 14 days.—Autolene Lubricants Co., Denver, Colo.

Cold process for manufacture of artificial marble and tile

In 1957 an Italian patent was obtained for a cold process of manufacturing artificial marble using approximately five parts cement to four parts stone dust with one part chemicals and colors. This process enables the production of both marble and tile in sheets of any color or form, adaptable for both interior and exterior finishes. Heat treat equipment or polishing machines are not required.

The representative for North America estimates manufacturing cost to be about 50 cents per sq yd of tile and $1 per sq yd of "marble," and states that both artificial marble and tile, cast in stainless steel forms, have a finish giving a highly polished effect.

Information and samples are available to those interested in utilizing this process.—Guido Martinez, 1118 Hamilton St., N.E., Washington, D. C.

Zinc-coated floor clips

Galvanized steel clip that anchors wood flooring to concrete floor decks is being marketed by Bull Dog Floor Clip Co.

Since tabs are set in moist concrete as first step in the floor laying operation, corrosion of the metal is a critical factor in service life. Galvanized steel was selected because of zinc's inherent resistance to corrosion.

After the concrete has hardened, the two top tabs are bent up, sleepers are placed between them, and nailed through prepunched holes in the tabs. Finished floor is nailed to sleepers in the usual manner. The clips are 0.040 in. thick, by 1 in. wide, and come in sizes to accommodate 2, 3, and 4-in. sleepers.—Bull Dog Floor Clip Co., 406 S.W. Ninth St., Des Moines 9, Iowa

CONCRETE TESTERS

The World's Finest Low-Cost Precision Testers

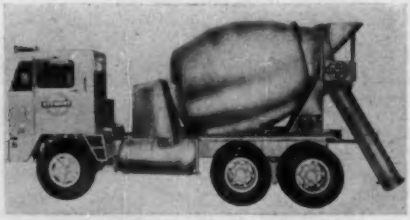
For
CYLINDERS
CUBES
BLOCKS
BEAMS
PIPE

IF IT'S A CONCRETE TESTER YOU NEED-GET IN TOUCH WITH

FORNEY'S, Inc.
TESTER DIVISION
P. O. BOX 310 - NEW CASTLE, PA.

Hydraulic powered concrete mixer

The 7-cu yd Stewart "Hydramix," by using a hydraulic oil pressure system to rotate the mixer, eliminates conventional mechanical linkages such as control



rods, lever, cranks, chains, and sprockets, according to AMPOZ (American Pozzolan Co.) They advise that this means greater simplicity of operation, reduced wear and expense, a shorter wheelbase, lower center of gravity, and less weight than with conventional mixers.

Three controls are mounted together on a panel on the rear support: a relay switch to control the drum, a hydraulic valve to position the chute, and a valve to control the water. Pressurized water system provides water for additions to the mix and for cleaning the drum and chutes. Unit is mounted on flywheel-take-off Crane Carrier truck, but mixing unit is also available separately for ready installation on all makes and models of trucks normally used.—AMPOZ, P. O. Drawer 1431, Duncan, Okla.

New, Lower Cost
Flexible
POST TENSION CASING

U-100WP
(Wide Profile)
ECONOFLEX
Ask for Bulletin U-100WP

Quality . . . ALL METAL
FLEXIBLE HOSE PRODUCTS
UNIVERSAL
METAL HOSE CO.
2101 S. Kedzie Ave., Chicago 23, Ill.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

FIVE MILES OF ENGINEERING INGENUITY... THE MACKINAC BRIDGE—illustrated brochure describing the design and construction of the bridge. Shows on-the-job photos of concrete being placed as well as photos of the bridge in various stages of construction.—Light Weight Aggregate Corp., 12720 Farmington Road, Livonia, Mich.

FIR PLYWOOD AND WESTERN SOFTWOOD PLYWOOD—three-part, 20-page catalog for 1958 presents basic information on fir plywood standard grades and specialty products. First section of general information includes tabulated data on plywood grades; FHA and building code requirements; engineering data for use of plywood for concrete forms; veneer descriptions; and quality control. The other two sections cover fir plywood properties for product design and descriptions of specialty products. Catalog is planned for quick reference with concise tables and illustrations of products and application.—Douglas Fir Plywood Association, Tacoma 2, Wash.

GILSON TESTING SCREEN—file folder of data sheets on testing screen and accessories for sizing test samples of crushed stone, sand and gravel, slag, coal and coke, ores, and similar materials. New sheets and pertinent information will be issued to keep this file current.—The Gilson Screen Co., 110 Center St., Malinta, Ohio

VERMICULITE CONCRETE ROOF INSULATION AND ROOF DECKS—eight data sheets and "long form" specifications provide details on proportioning, mixing, and structural design. Four are data sheets on vermiculite concrete roof insulation, and an equal number cover vermiculite concrete roof decks.—Vermiculite Institute, 208 South LaSalle St., Chicago 4, Ill.

SUPER-LIFT SCREW ELEVATOR WITH DRY SHAFT DRIVE—brochure describing screw elevator with dry shaft drive for handling free-flowing and many sluggish bulk materials. Includes cross section drawings of units and detailed description of how dry shaft drive proves triple protection against oil leakage.—Fort Worth Steel and Machinery, P. O. Box 1038, Fort Worth, Tex.

SURFONIC SURFACE-ACTIVE AGENTS TECHNICAL BULLETIN, 1958—40-pages about adducts of nonylphenol or tridecyl alcohol with ethylene oxide. Manual provides available information on analytical procedures, physical and surface-active properties, and typical industrial uses of these nonionic surfactants; a typical sulfation procedure is a feature of the book. Surface-active agents of the type described in this bulletin find extensive use in the preparation of concrete additives.—Jefferson Chemical Co., Inc., 1211 Walker Ave., Houston 2, Tex.

Plastiment Concrete Densifier

Cuts Construction Time

ON HARTFORD'S WINDSOR BRIDGE

▲ DECK CONCRETE IS PLACED AND VIBRATED INTO A MONOLITHIC MASS WITH BEAMS.

14 PAIRS OF 120 FOOT SPANS WERE CONCRETED IN 99 DAYS. ▼

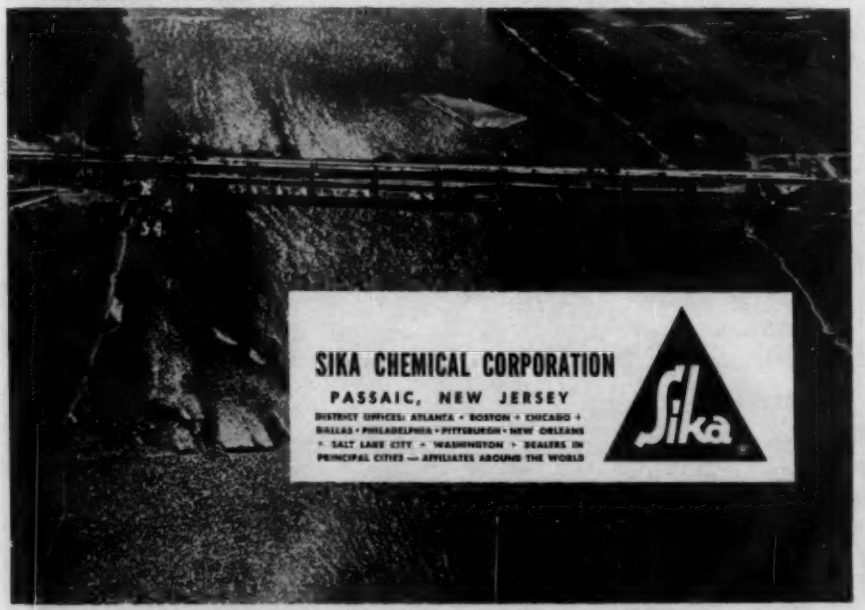
Rapid construction of concrete spans on this 1677 foot bridge across the Connecticut River came as no surprise to the contractor and consulting engineers. . . . Using this new procedure, these monolithic cast-in-place prestressed concrete spans were completed in a record 99 days. . . . This new monolithic beam and slab construction procedure was made possible by controlling the set of concrete with Plastiment Concrete Densifier.

The contractor increased the Plastiment proportion for the 7 foot deep beams which were placed in the morning. . . . After lunch this concrete was further consolidated by revibration. . . . Then the deck slab concrete containing a normal Plastiment proportion was placed and carefully vibrated into a monolithic mass with the beams. . . . Although initial set was retarded, strength developed very rapidly to 3000 psi within four days. . . . Prestressing operations were begun, forms removed, and in less than 10 days 20-ton truck mixers were moving across the span to place other spans.

The Plastiment characteristics of set control and rapid development of high strength make possible considerable savings in construction time. . . . It also provides an additional bonus of structural benefits including reduced cracking, increased density and higher bond to steel. . . . Let Plastiment save time and dollars for you. . . . For complete information write for Bulletin PCD.

Greater Hartford Bridge Authority—Owner
The Preload Company, Inc., N. Y. C.—Consulting Engineers for Prestressed Design and Construction
Merritt-Chapman & Scott, N. Y. C.—Contractor
DeLeuw Cather & Brill, N. Y. C.—Consulting Engineers for Owner
Thomas Worcester, Boston Mass.—Design Consultants

Ad 26-10


SIKA CHEMICAL CORPORATION
PASSAIC, NEW JERSEY
DISTRICT OFFICES: ATLANTA • BOSTON • CHICAGO • DALLAS • PHILADELPHIA • PITTSBURGH • NEW ORLEANS • SALT LAKE CITY • WASHINGTON • DEALERS IN PRINCIPAL CITIES — AFFILIATES AROUND THE WORLD
Sika

ALPHABETICAL LIST OF ADVERTISERS

(Page Numbers refer to News Letter)

Autolene Lubricants Company....26

Colorado Fuel and Iron Corporation, Clinton Division....18

Concrete Reinforcing Steel Institute....14

Forney's, Inc....31

Lehigh Portland Cement Co....12

Lone Star Cement Corporation....Inside Front Cover

Marathon....21

Master Builders Co., The....22

Sika Chemical Corporation....33

Techkote Company, Incorporated....16

Universal Metal Hose Co....32

Western Waterproofing Co., Inc....25

The Institute assumes no responsibility for the claims of advertisers. The advertiser is made responsible in the belief that his place in the field will be determined by the public's ultimate measure of his exercise of that responsibility.

NOTICE — Change of Address — NOTICE

To avert any delay in receiving my ACI JOURNAL, I wish to give notice of a change in my mailing address. (PLEASE PRINT)

New Address:

NAME_____

STREET & NO._____

CITY_____ ZONE____ STATE_____

Old Address:

STREET & NO._____

CITY_____ ZONE____ STATE_____

Detailing Practices

Buildings to Bridges

58 Illustrative Drawings

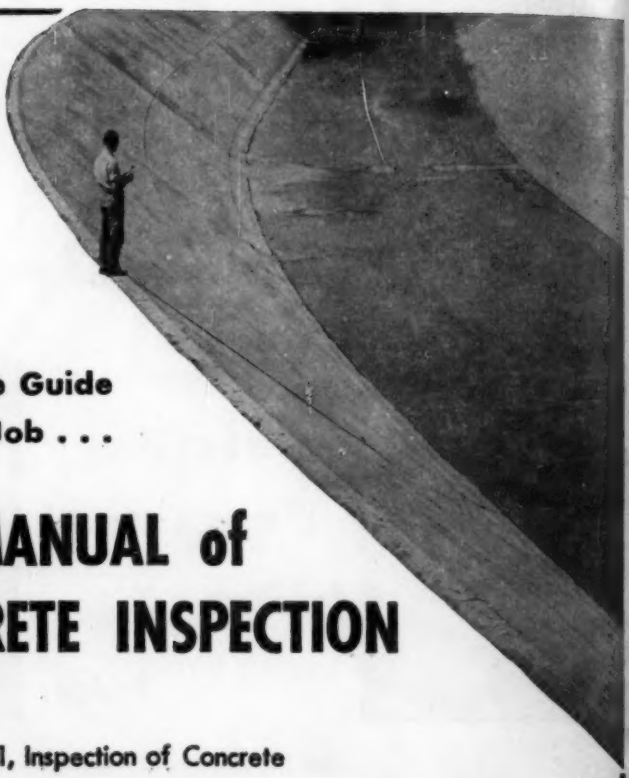
The ***Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)*** incorporates under one cover the previous separate editions on detailing of buildings and highway structures. It is a correlation of the latest improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. Sections on detailing and fabricating shop practice are translated into practical examples in typical drawings. Spiral bound to lie flat, it is the only publication of its kind in English and is meeting wide acclaim among designers, draftsmen, and engineering schools.

Full Price $4.00 **ACI Members** $2.50

AMERICAN CONCRETE INSTITUTE

PUBLICATIONS

P.O. Box 4754, Redford Station Detroit 19, Mich.



Pocketsize Guide to a Big Job . . .

ACI MANUAL of CONCRETE INSPECTION

Third Edition

Committee 611, Inspection of Concrete

A comprehensive handbook with both the "why" and "how" of inspection, in convenient durable form.

From concrete fundamentals to the latest developments in construction, the manual covers problems and techniques in concrete inspection clearly and completely. Written and bound for use at the construction site as well as the laboratory and design office, the manual is a key tool for the concrete inspector and a valuable aid to anyone responsible for workmanship in concrete.

(Price $3.50—ACI Members $1.75)

AMERICAN CONCRETE INSTITUTE **PUBLICATIONS**

P.O. Box 4754, Redford Station Detroit 19, Mich.